REPORT APPENDIX

IDENTIFICATION OF CAUSES AND DEVELOPMENT OF STRATEGIES FOR RELIEVING STRUCTURAL DISTRESS IN BRIDGE ABUTMENTS

by

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A. Residual Plots of Linear Regression Models

The residual plots listed in this section were derived for linear regression model considering second term of explanatory variables (section 3.6 of the final report). The plot of residual against length is shown Figure A-1. It can be seen that there is structural pattern in the residual plot and that the variance of the residual is not constant. There are some outliers in the plot, especially when the length values are small.

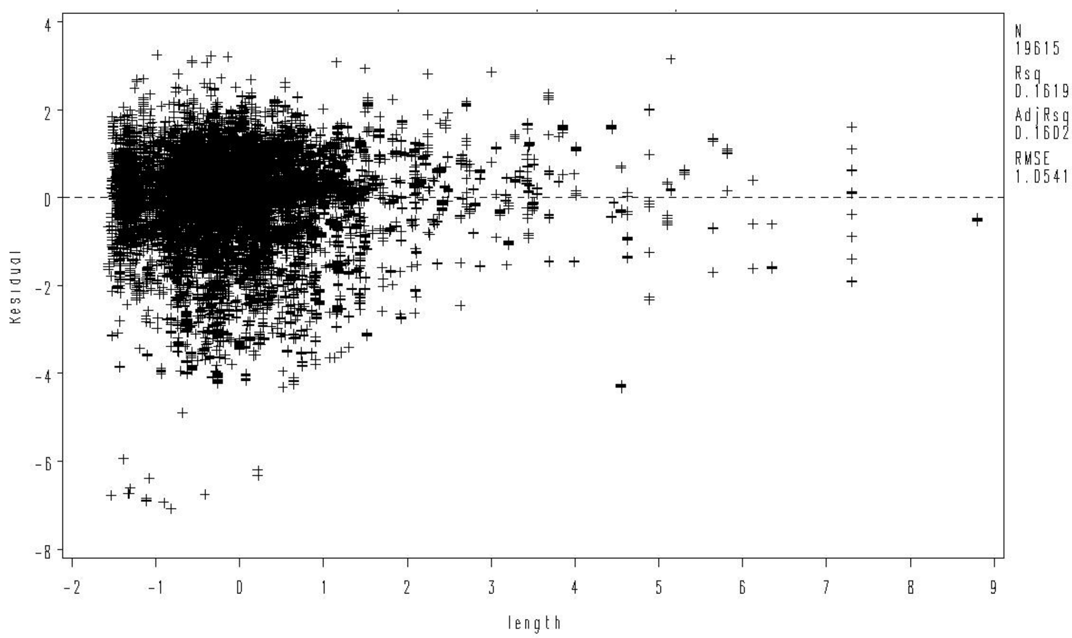


Figure A-1 Plot of Residual against length

The plot of residual against age at inspection is shown in Figure A-2. It can be seen that there is structural pattern in the residual plot. Caution shall be taken that the there are some outliers in the plot, even though the problem is not severe since most of the residuals fall within the range of ±2. The plot of residual against average annual temperature difference is shown in Figure A-3. No systematic pattern has been found from Figure A-3.

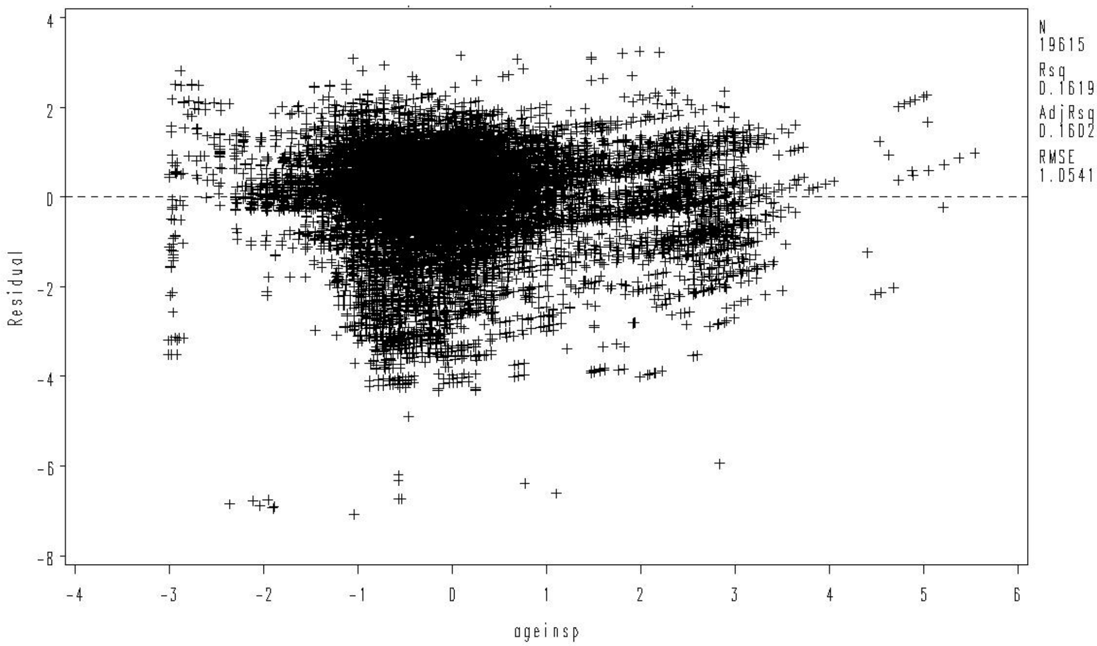


Figure A-2 Plot of Residual against ageinsp

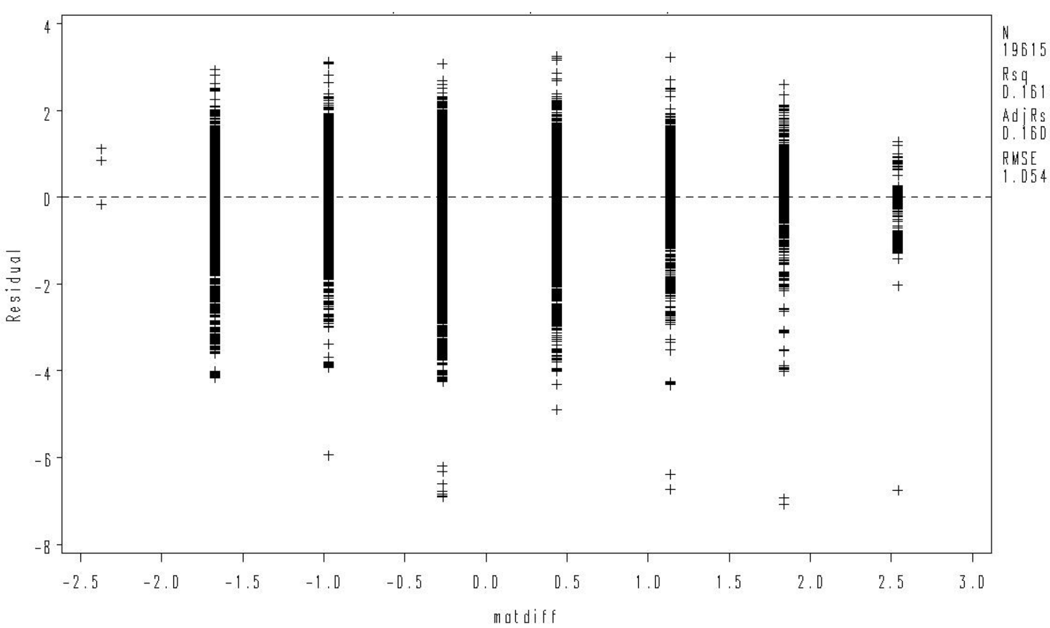


Figure A-3 Plot of Residual against matdiff

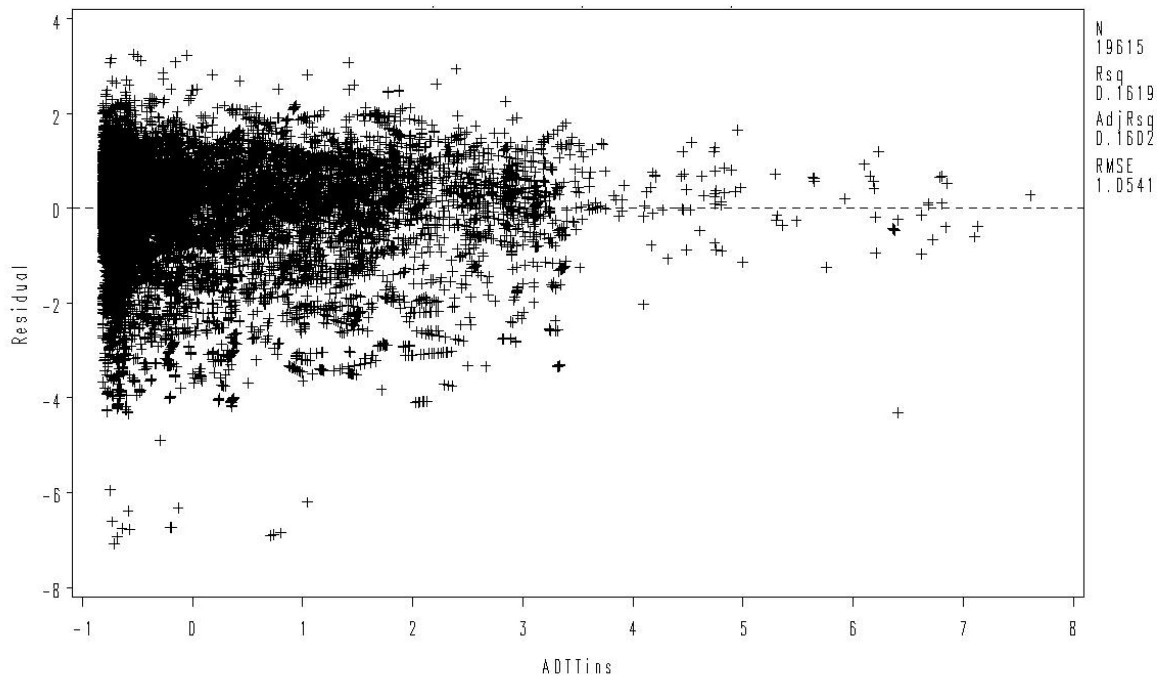


Figure A-4 Plot of Residual against ADTTins

The plot of residual against average daily truck traffic at inspection (ADTT) is shown in Figure A-4. A fan shape has been noticed, the variance of the residual tends to decrease with the increase of ADTT. The plot of residual against maximum span is shown in Figure A-5 and no obvious deficiency can be found out from the plot.

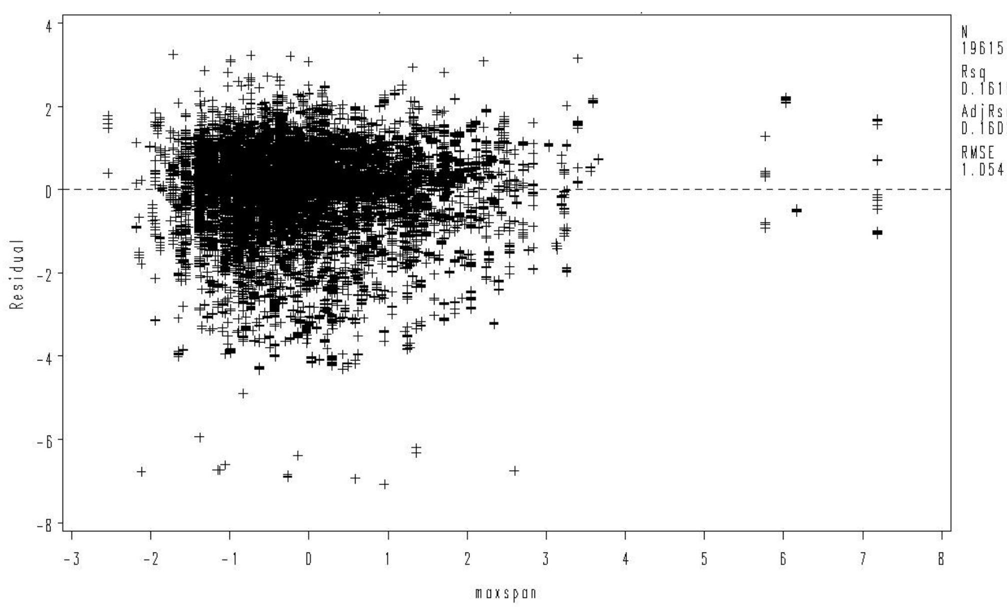


Figure A-5 Plot of Residual against maxspan

B. Design Plan and Bearing Details of Instrumented Bridges

In Task II of this research project, the bridges chosen to be instrumented are required to have pin-dowel connection between girder and abutment wall. The design plans and bearing details of candidate bridges were reviewed to make sure that the instrumented bridges meet the requirements. The design plan and bearing details of instrumented bridges are shown in Figure B-1 to Figure B-10.

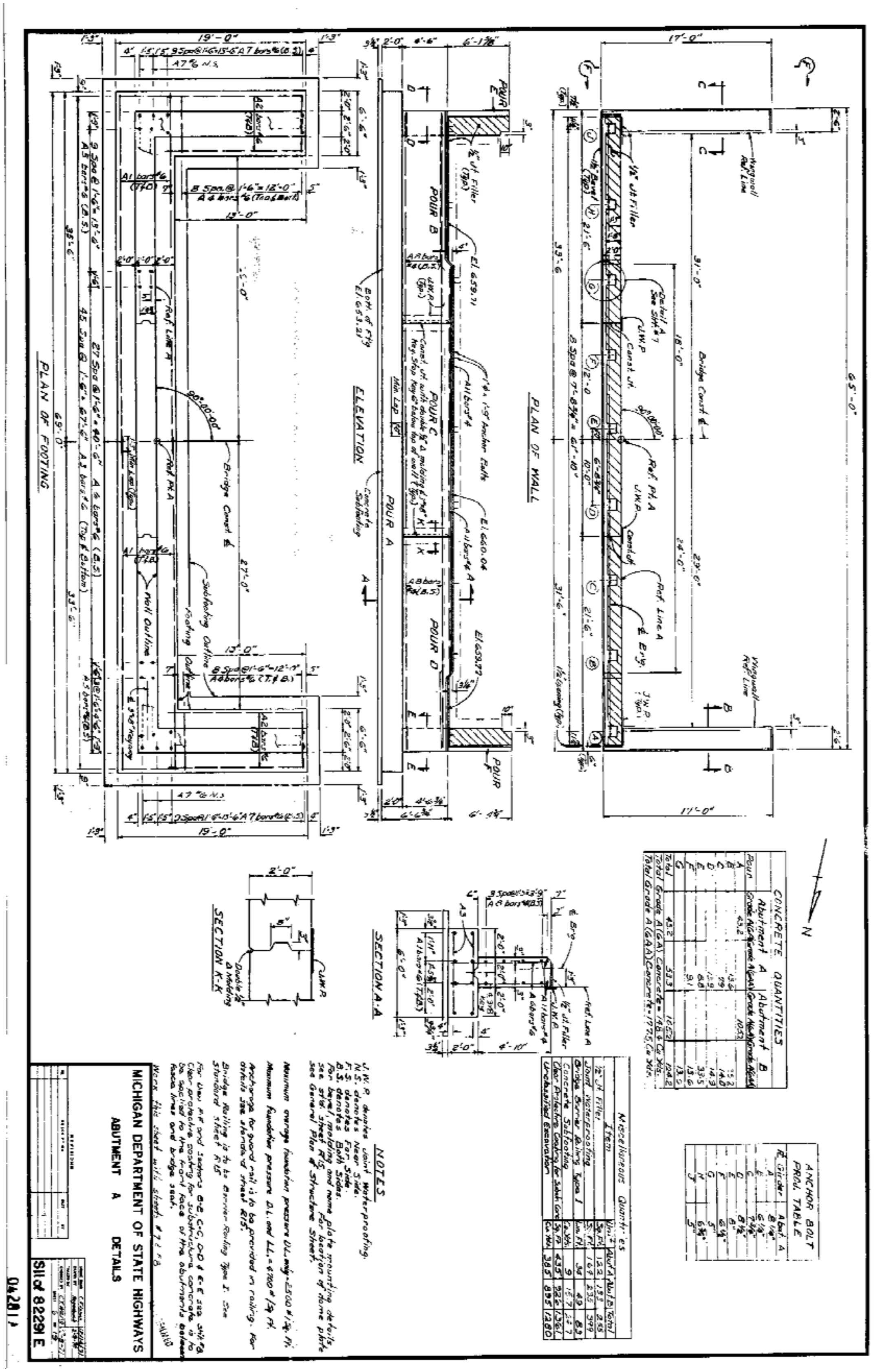


Figure B-1 Abutment A details of bridge A 1.7

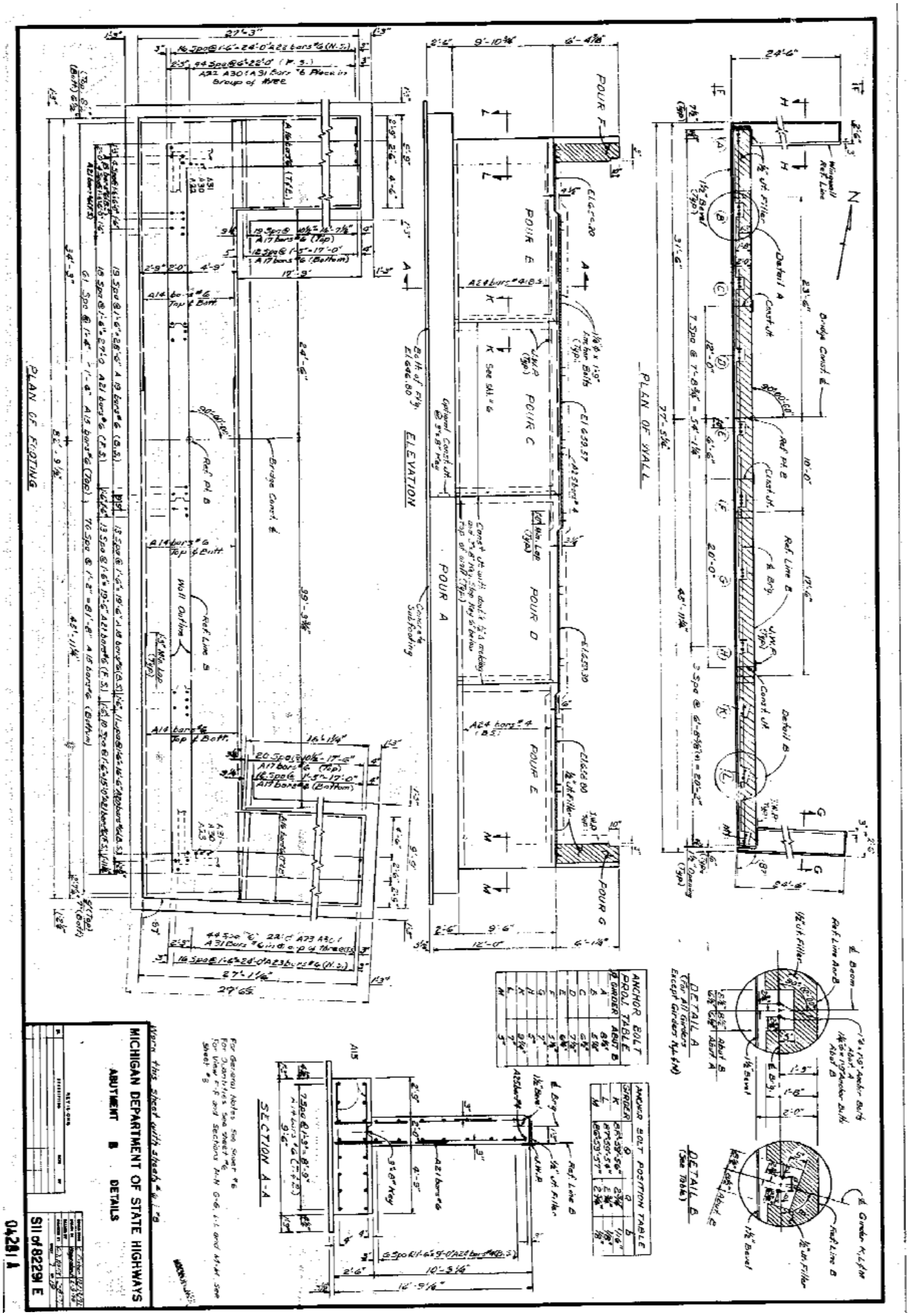


Figure B-2 Abutment B details of bridge A 1.7

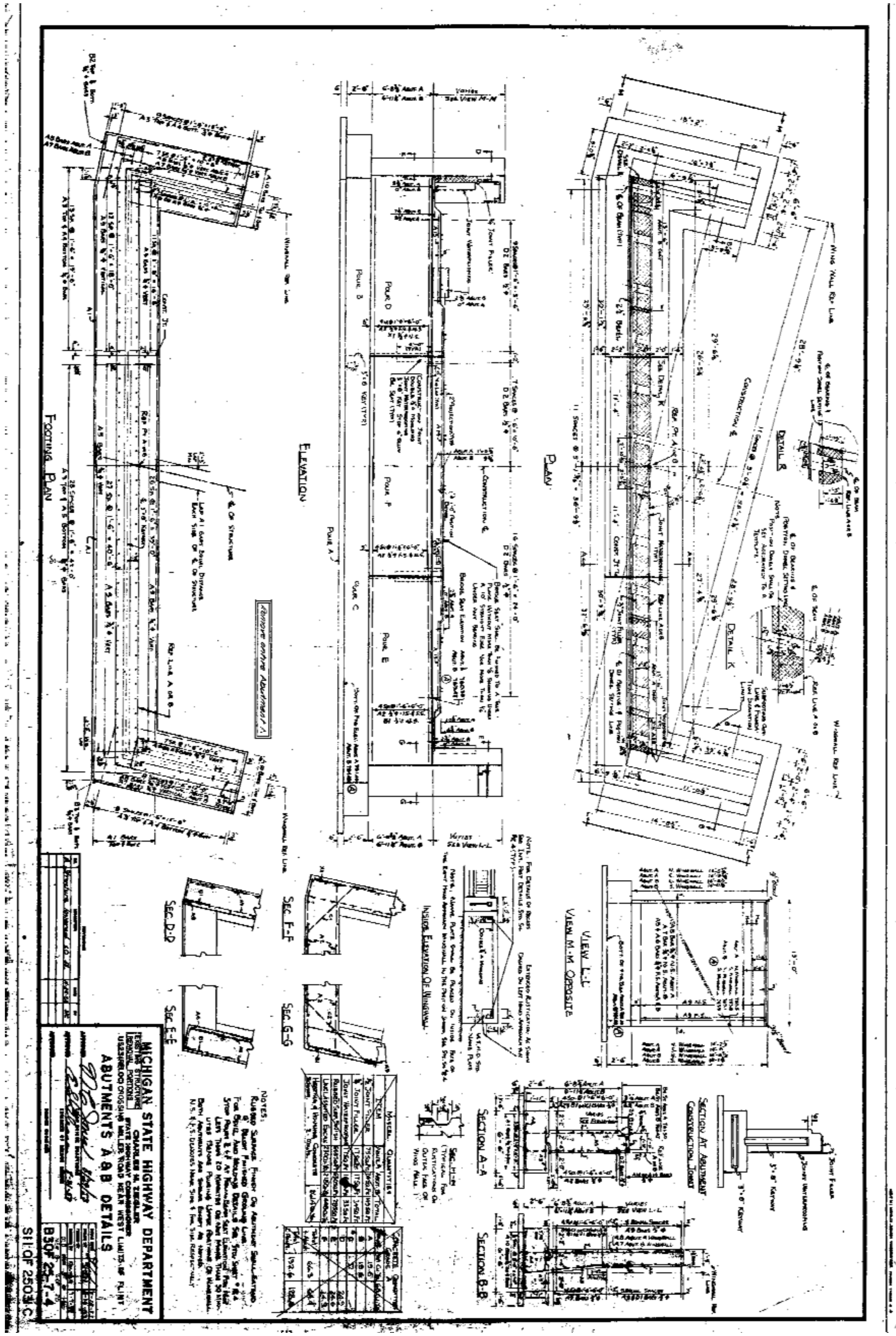


Figure B-3 Abutment A & B details of bridge A 2.1

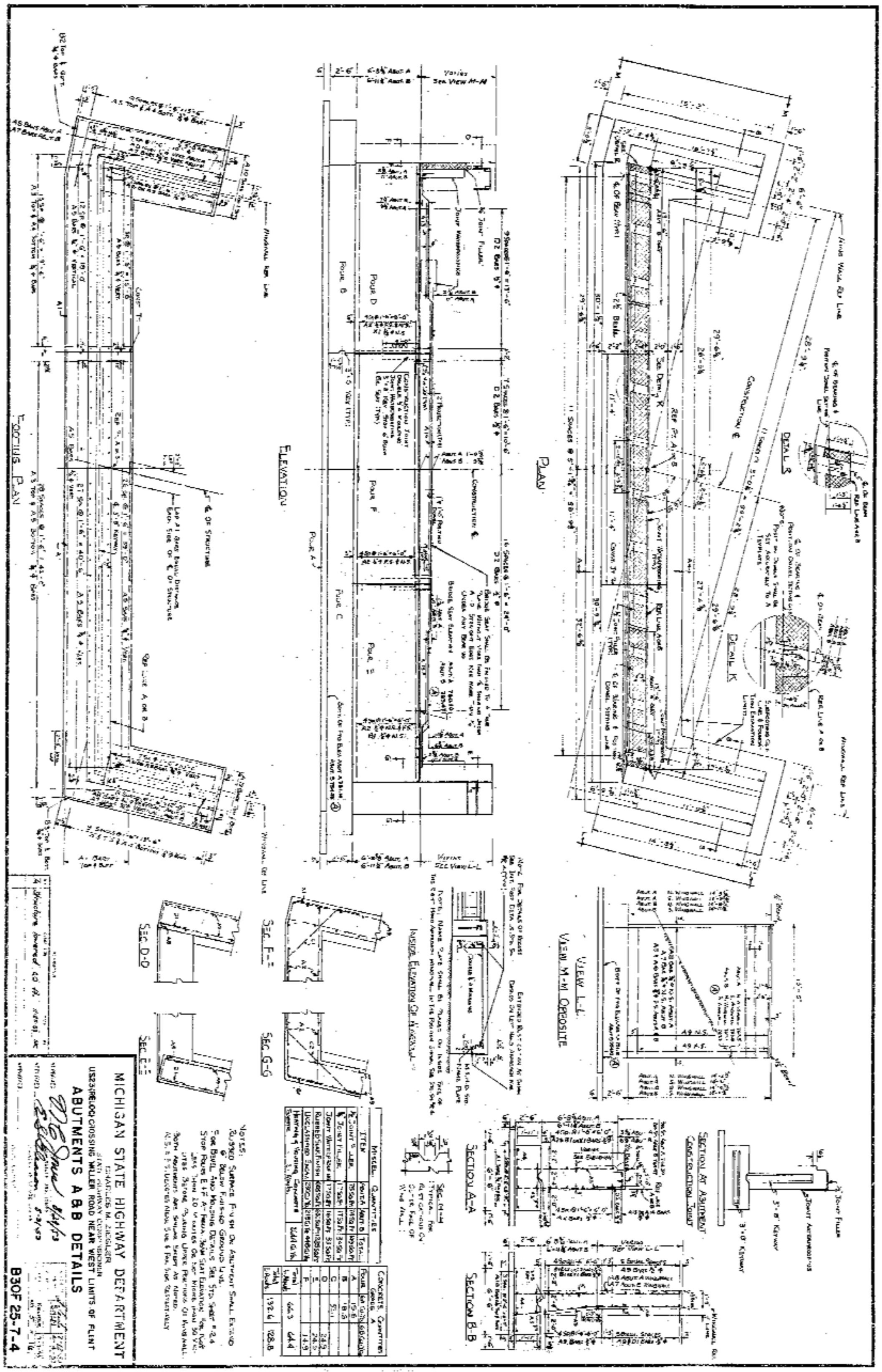


Figure B-4 Abutment A & B details of bridge A 2.1

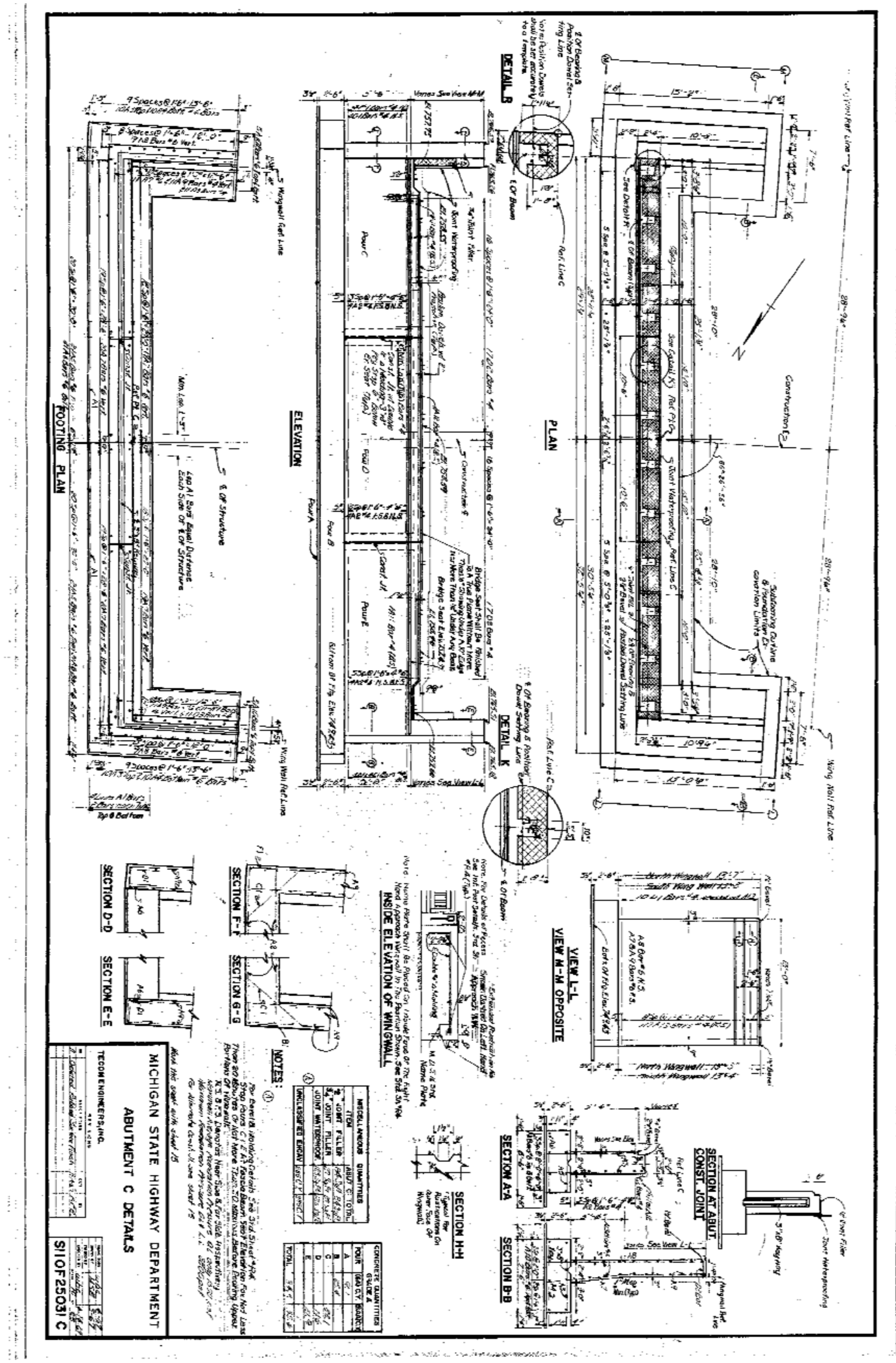


Figure B-5 Abutment C details of bridge A 2.1

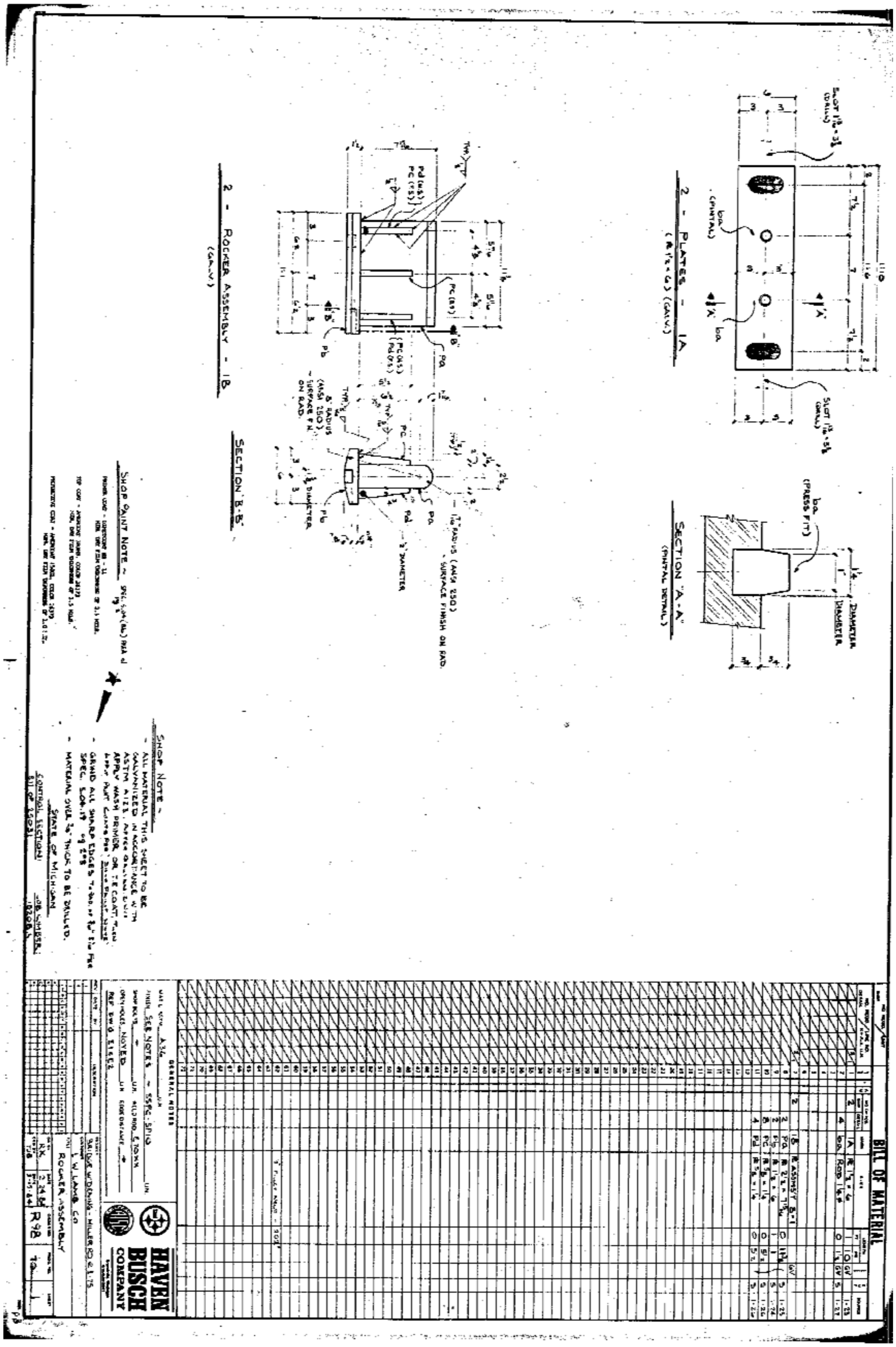


Figure B-6 Rocker details (a) of bridge A 2.1

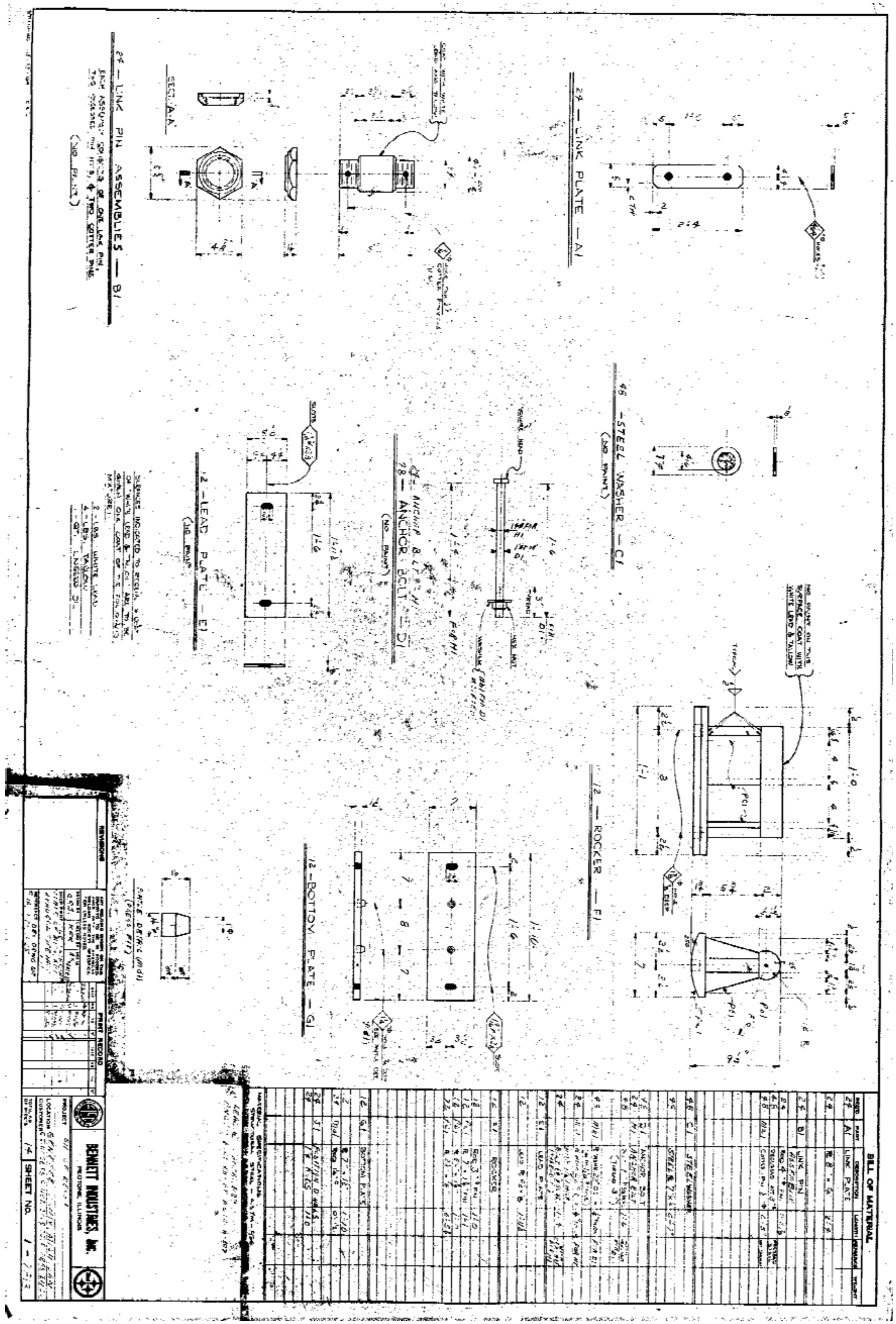


Figure B-7 Rocker details (b) of bridge A 2.1

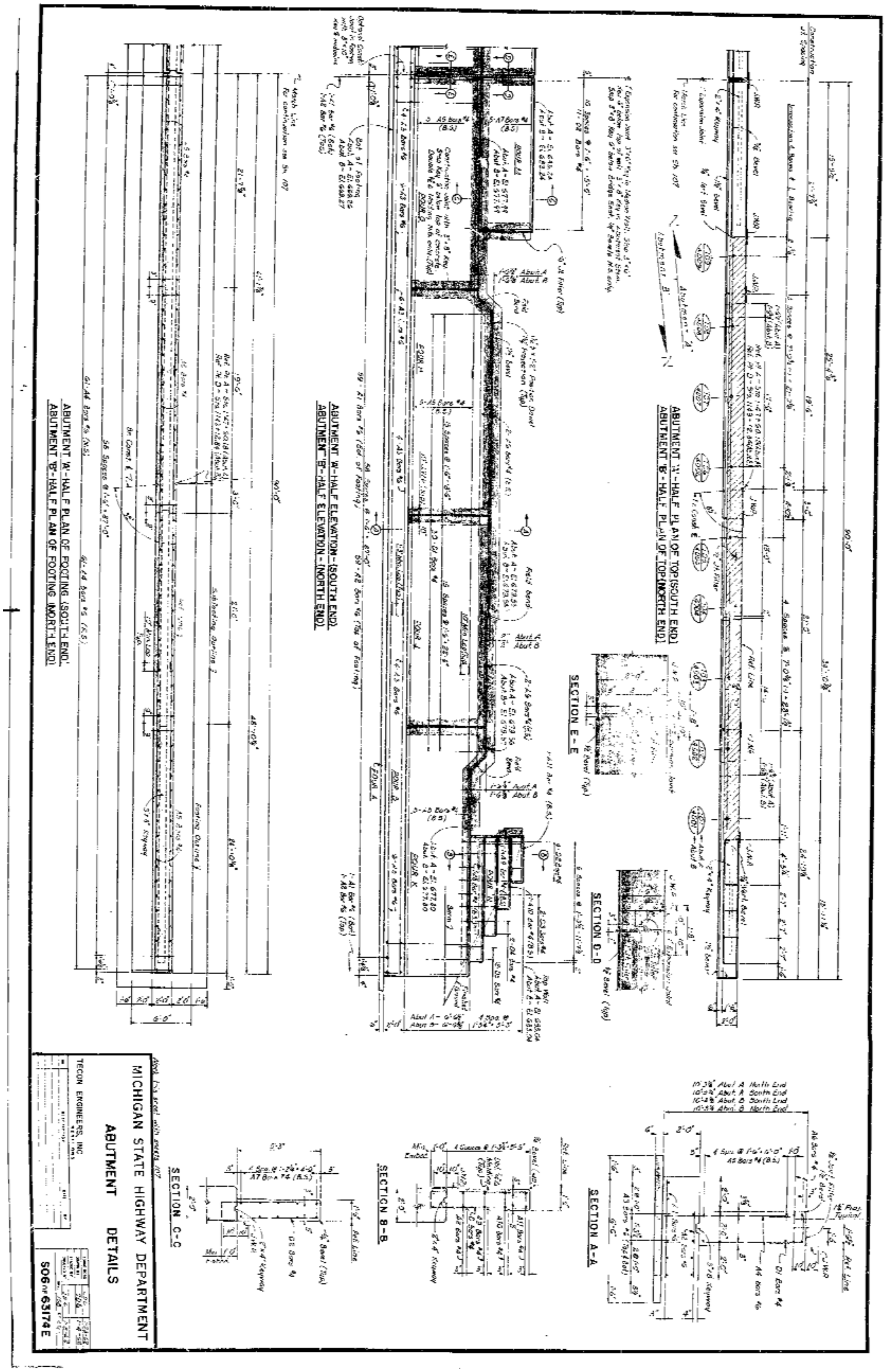


Figure B-8 Abutment details of bridge C 2.1

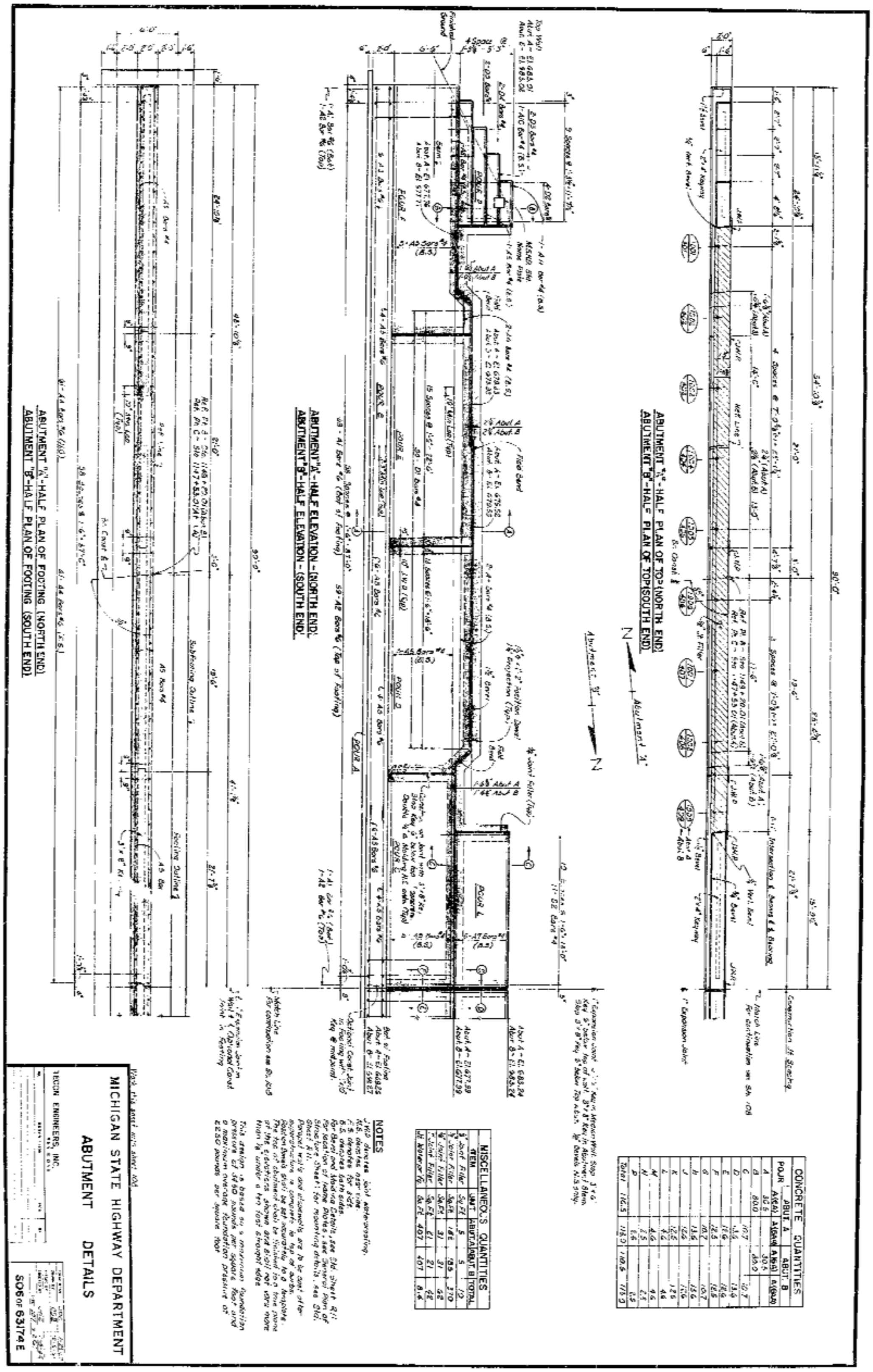


Figure B-9 Abutment A details of bridge C 2.1

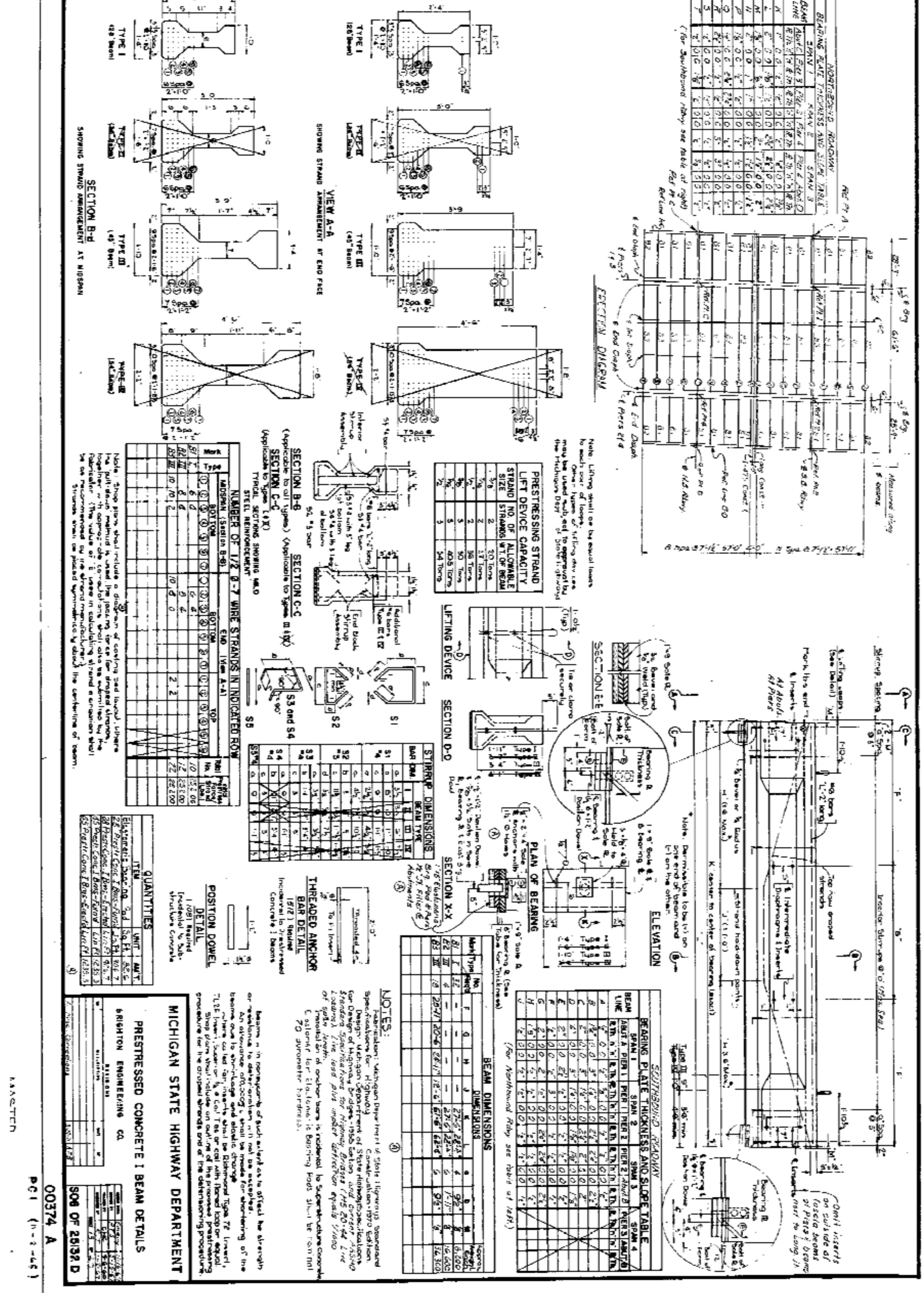


Figure B-10 Prestressed concrete I beam details of bridge C 2.4

C. Analyses of Field Instrumentation Data

All the figures created during the analysis of field data are listed as follows:

C.I Distribution of Strains

C.I.1 Distribution of strains along abutment walls

The distributions of horizontal strains in the abutment wall of bridge A 1.7 are shown in Figure C-1 to Figure C-11. Similarly, the distributions of horizontal strains in the abutment walls of bridges A 2.1, C 2.1, and C 2.4 are shown in **Figure** C-12 to **Figure** C-22, **Figure** C-23 to **Figure** C-33, and **Figure** C-34 to **Figure** C-44; respectively.

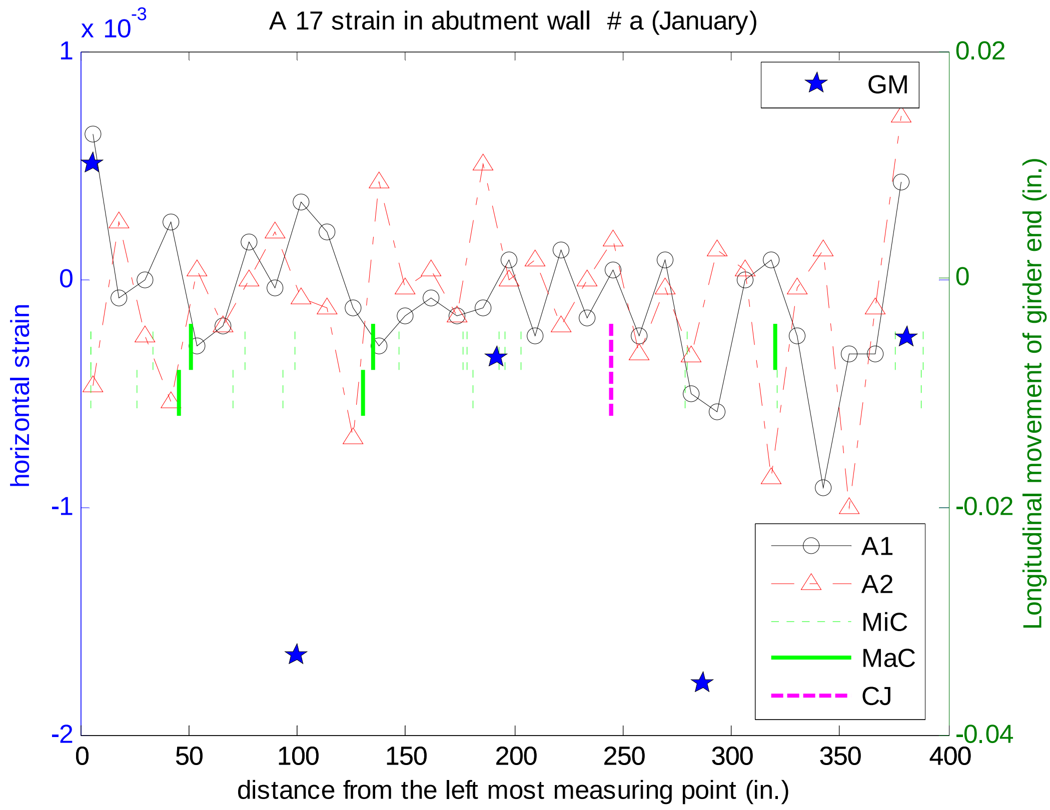


Figure C-1 Distribution of horizontal strain in abutment wall of Bridge A 1.7 in January 2007

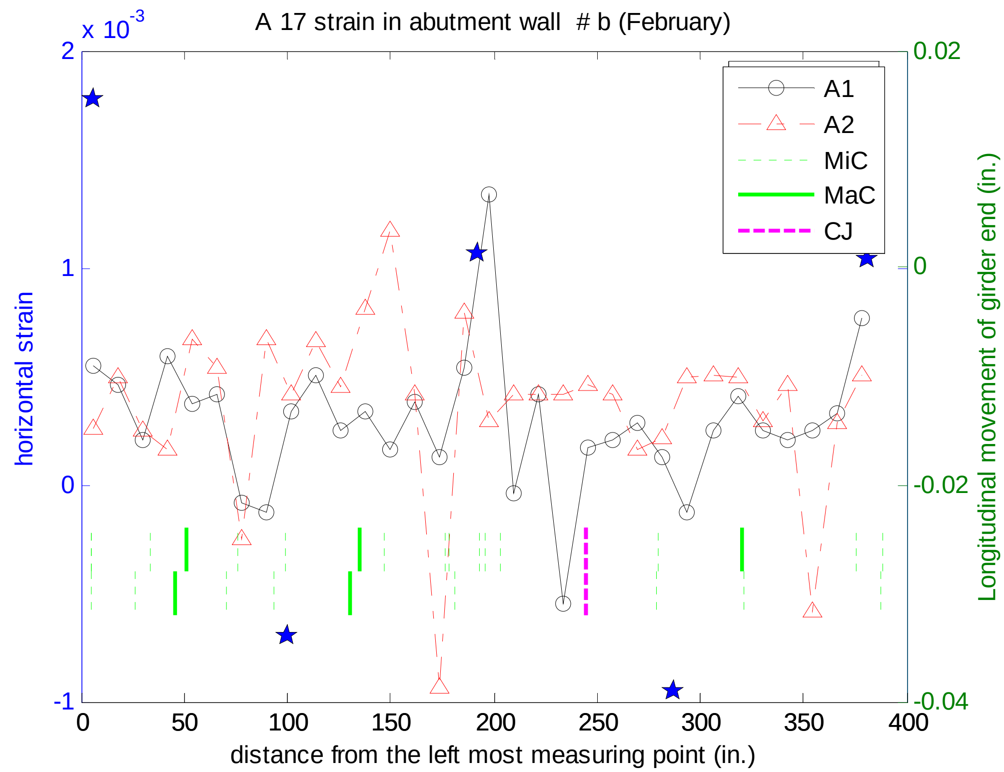


Figure C-2 Distribution of horizontal strain in abutment wall of Bridge A 1.7 in February 2007

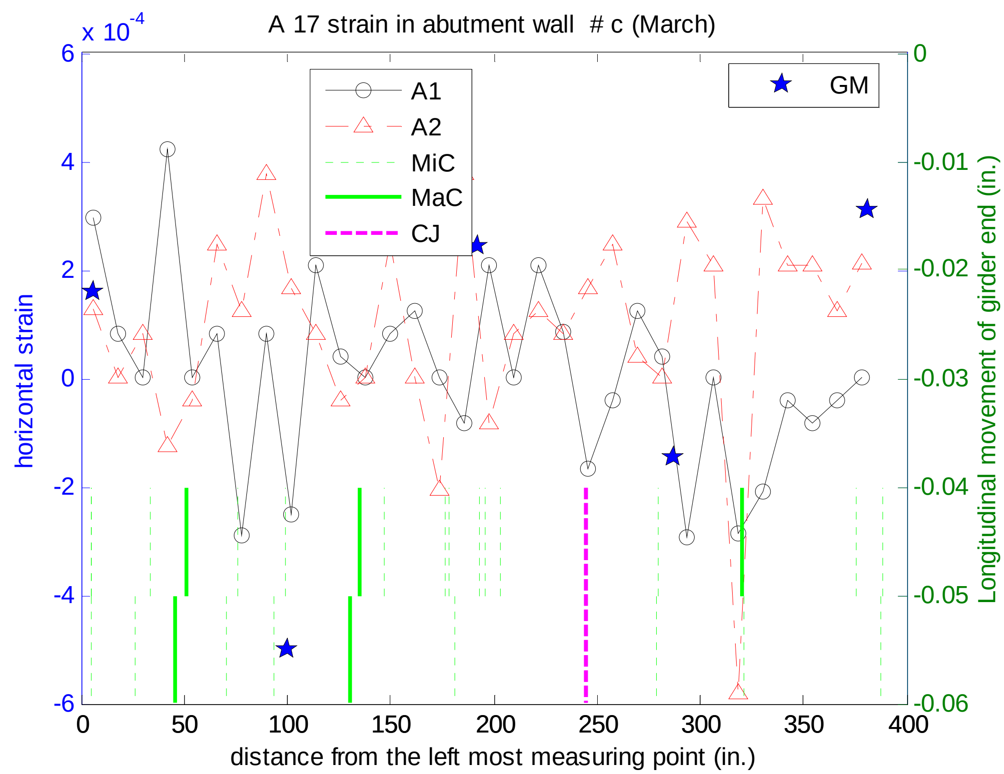


Figure C-3 Distribution of horizontal strain in abutment wall of Bridge A 1.7 in March 2007

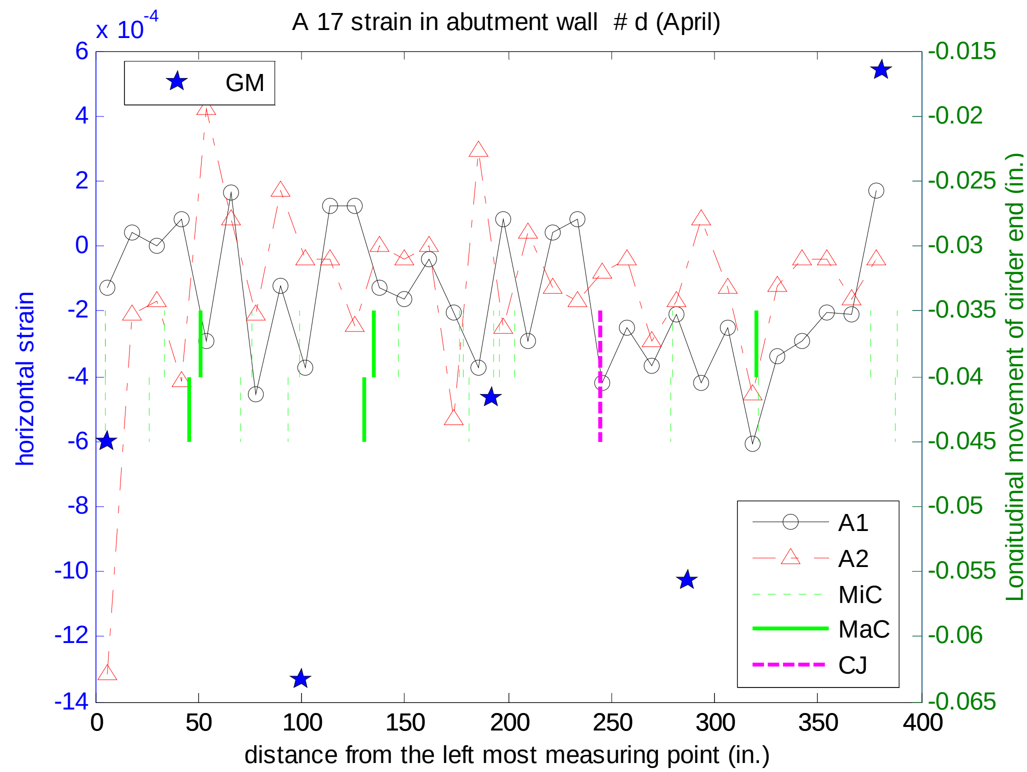


Figure C-4 Distribution of horizontal strain in abutment wall of Bridge A 1.7 in April 2007

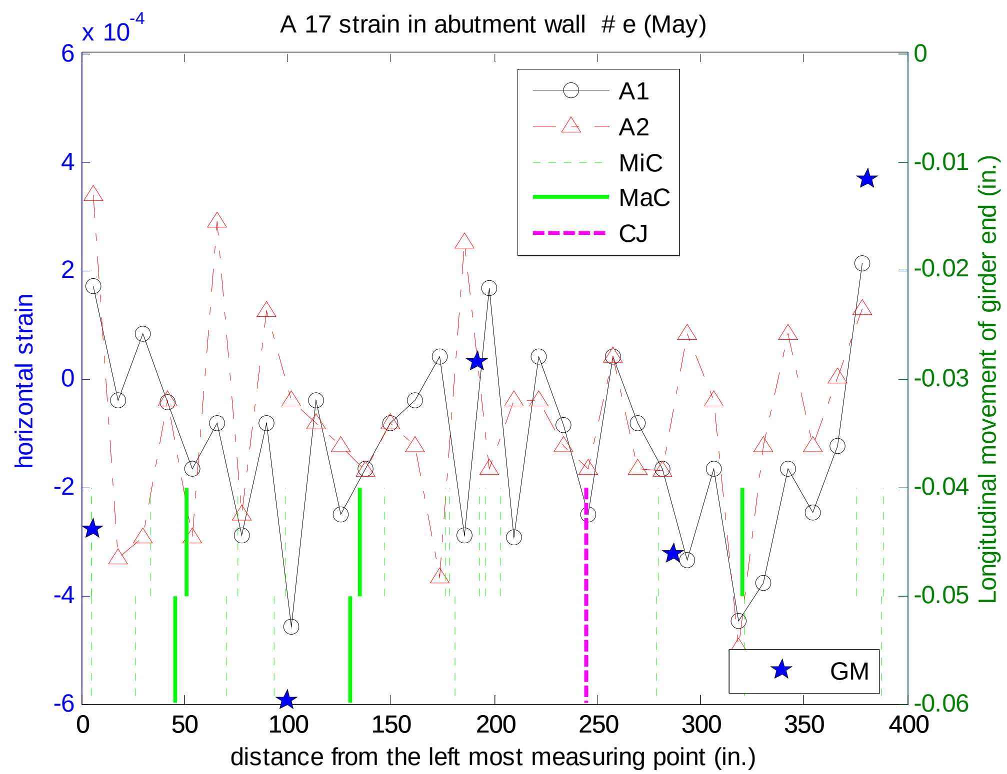


Figure C-5 Distribution of horizontal strain in abutment wall of Bridge A 1.7 in May 2007

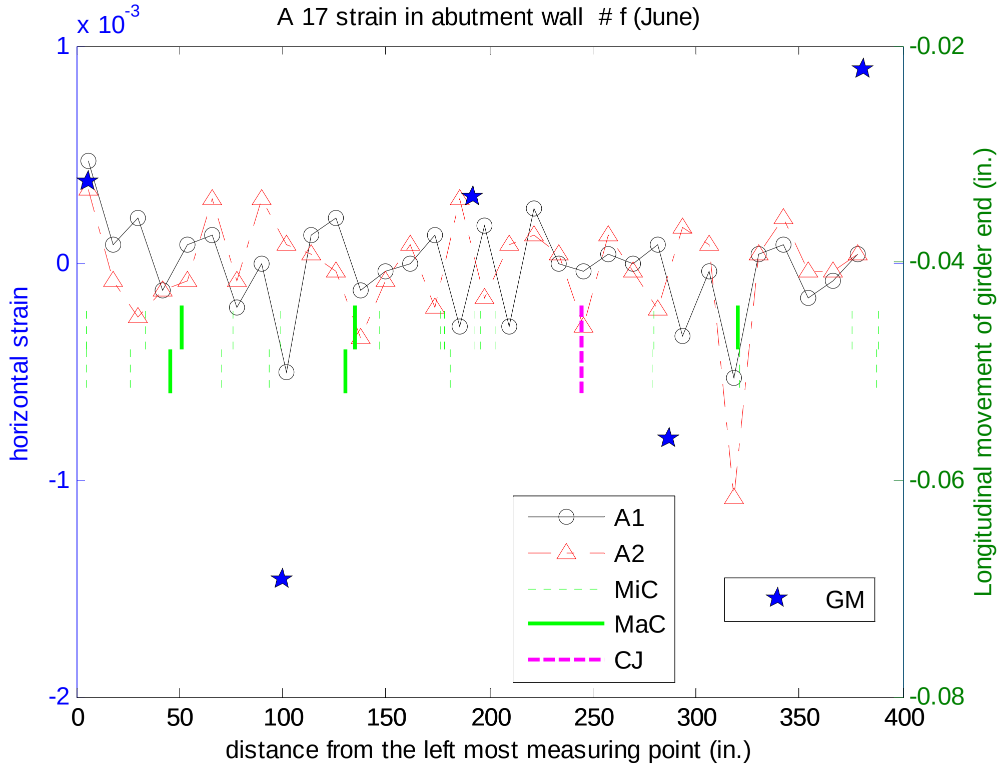


Figure C-6 Distribution of horizontal strain in abutment wall of Bridge A 1.7 in June 2007

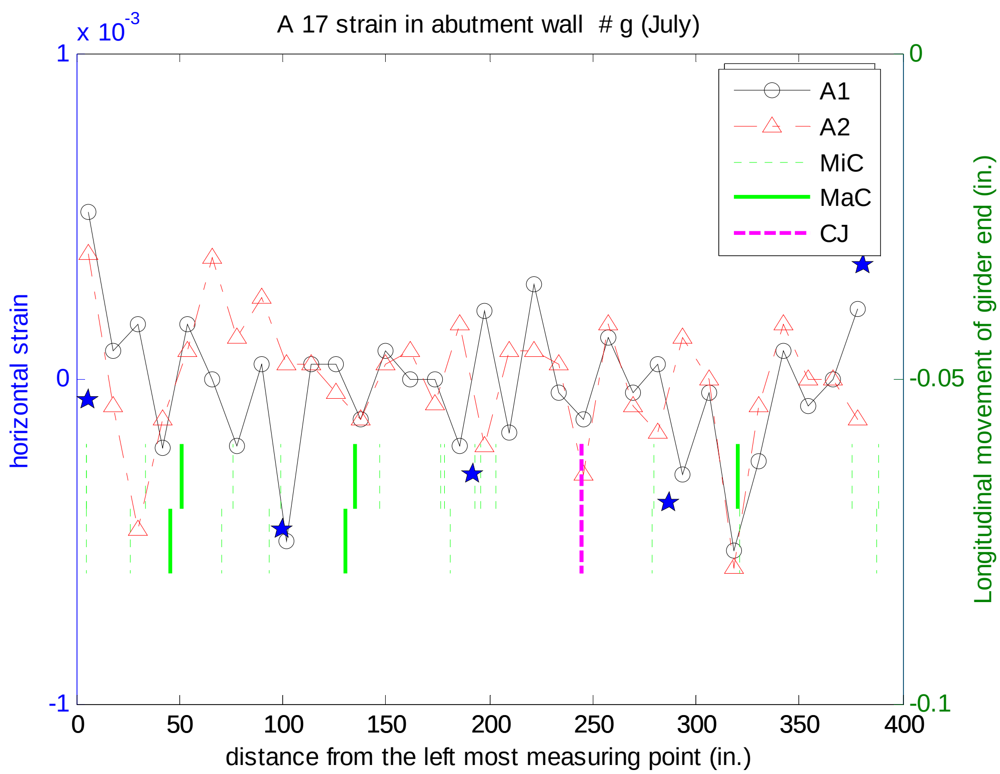


Figure C-7 Distribution of horizontal strain in abutment wall of Bridge A 1.7 in July 2007

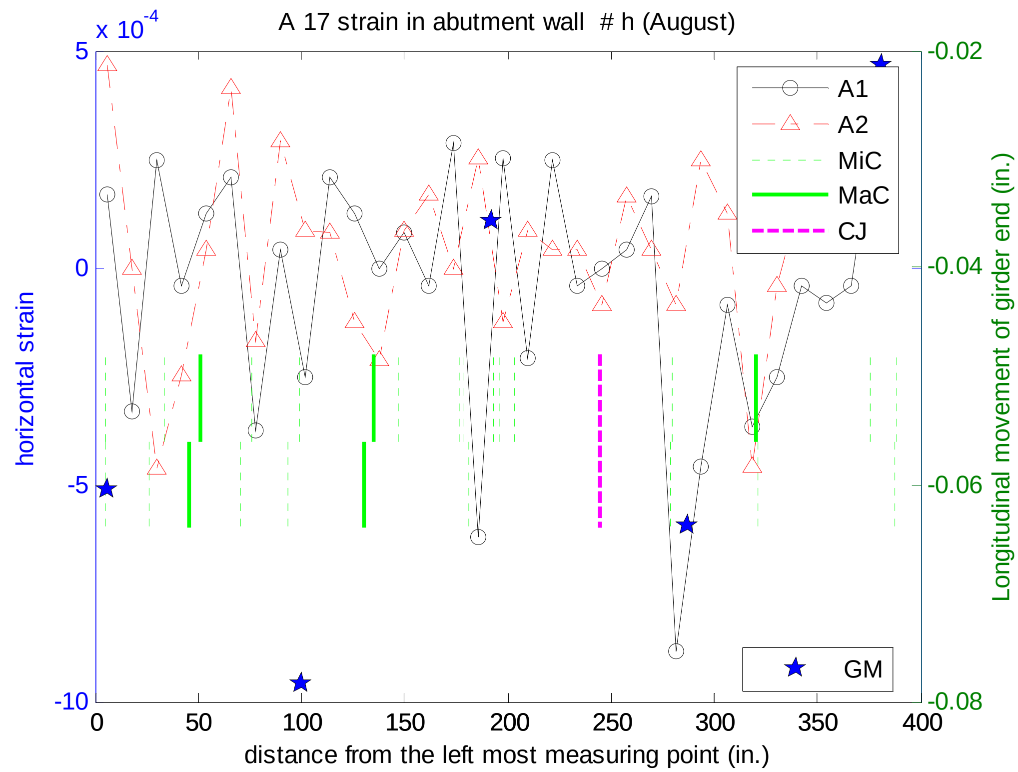


Figure C-8 Distribution of horizontal strain in abutment wall of Bridge A 1.7 in August 2007

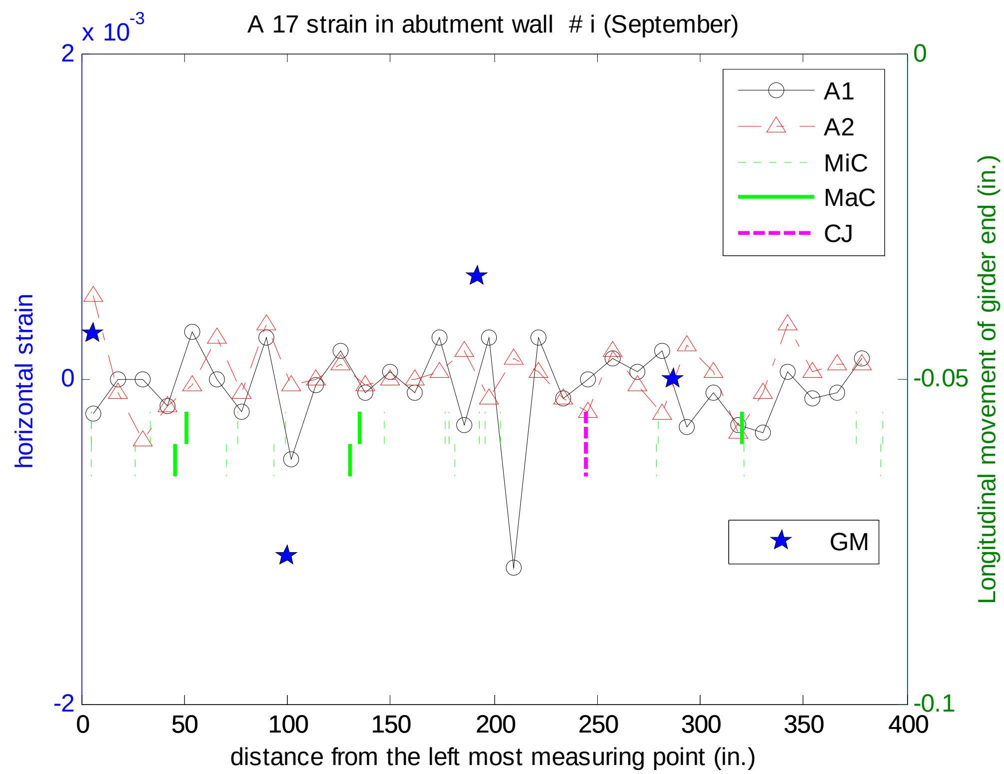


Figure C-9 Distribution of horizontal strain in abutment wall of Bridge A 1.7 in September 2007

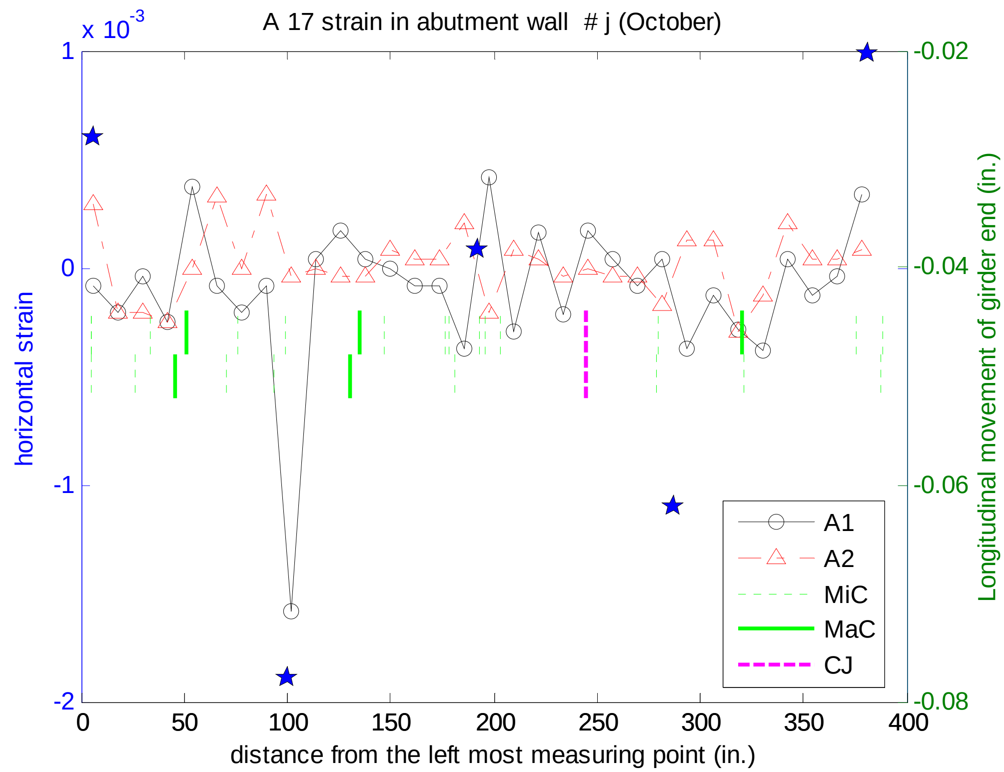


Figure C-10 Distribution of horizontal strain in abutment wall of Bridge A 1.7 in October 2007

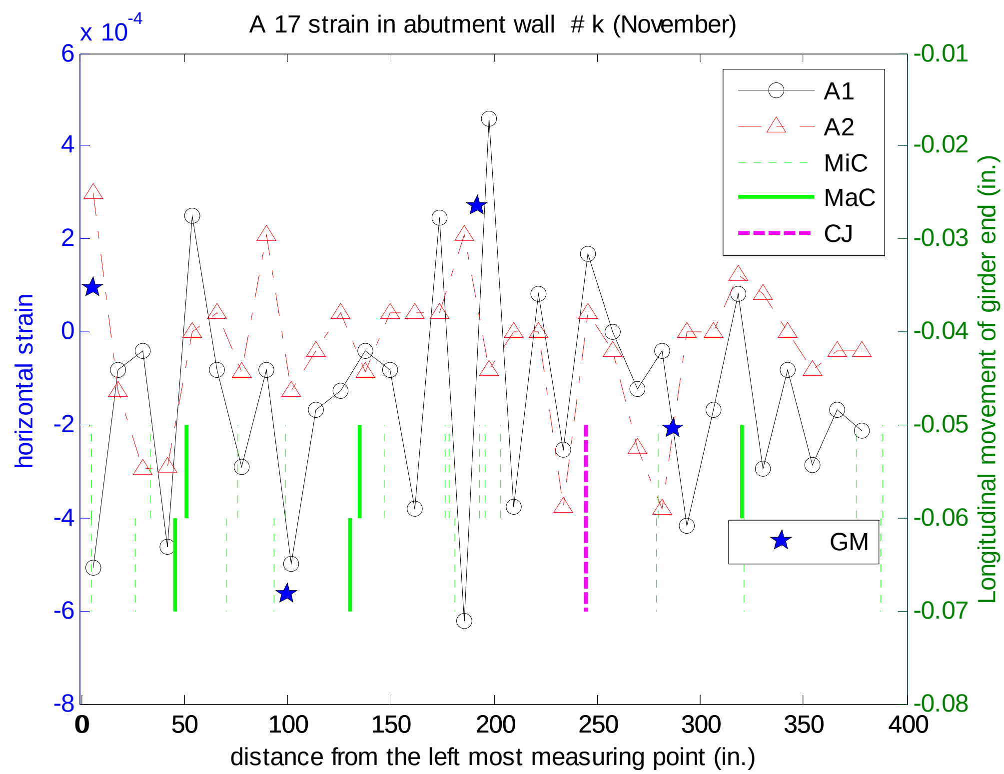


Figure C-11 Distribution of horizontal strain in abutment wall of Bridge A 1.7 in November 2007

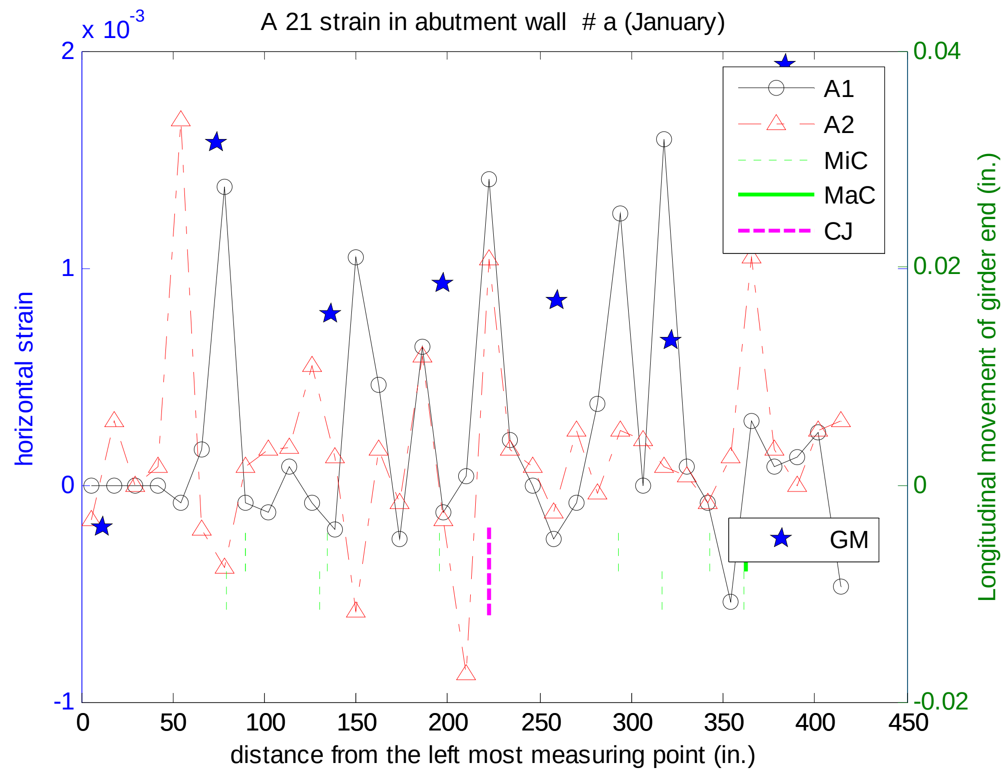


Figure C-12 Distribution of horizontal strain in abutment wall of Bridge A 2.1 in January 2007

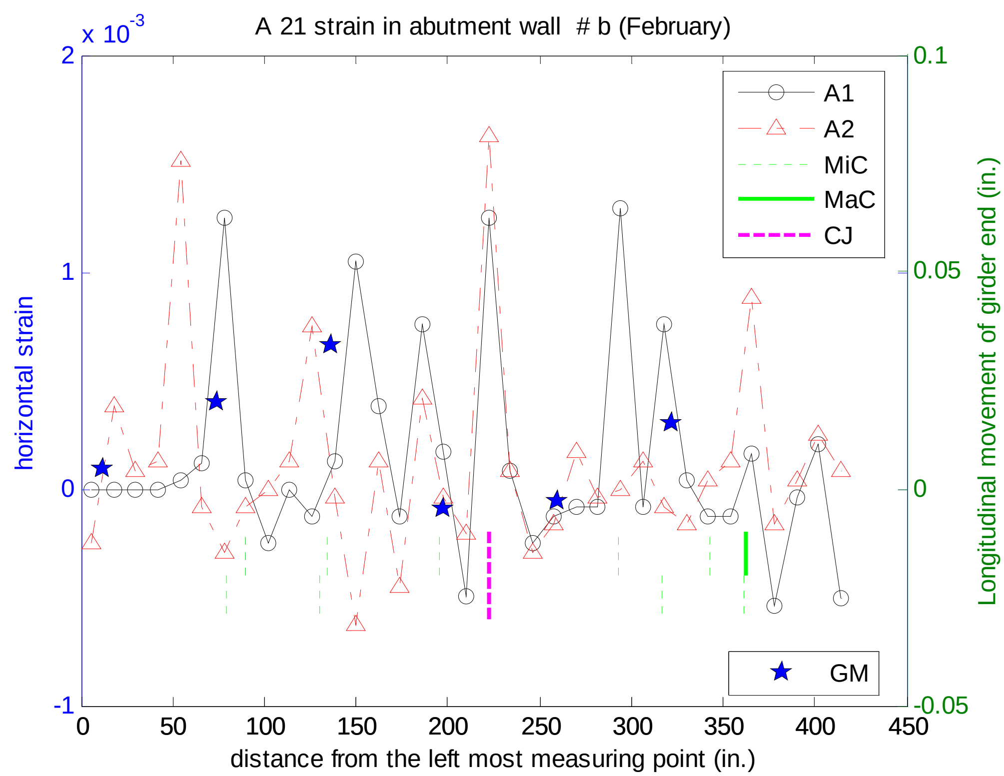


Figure C-13 Distribution of horizontal strain in abutment wall of Bridge A 2.1 in February 2007

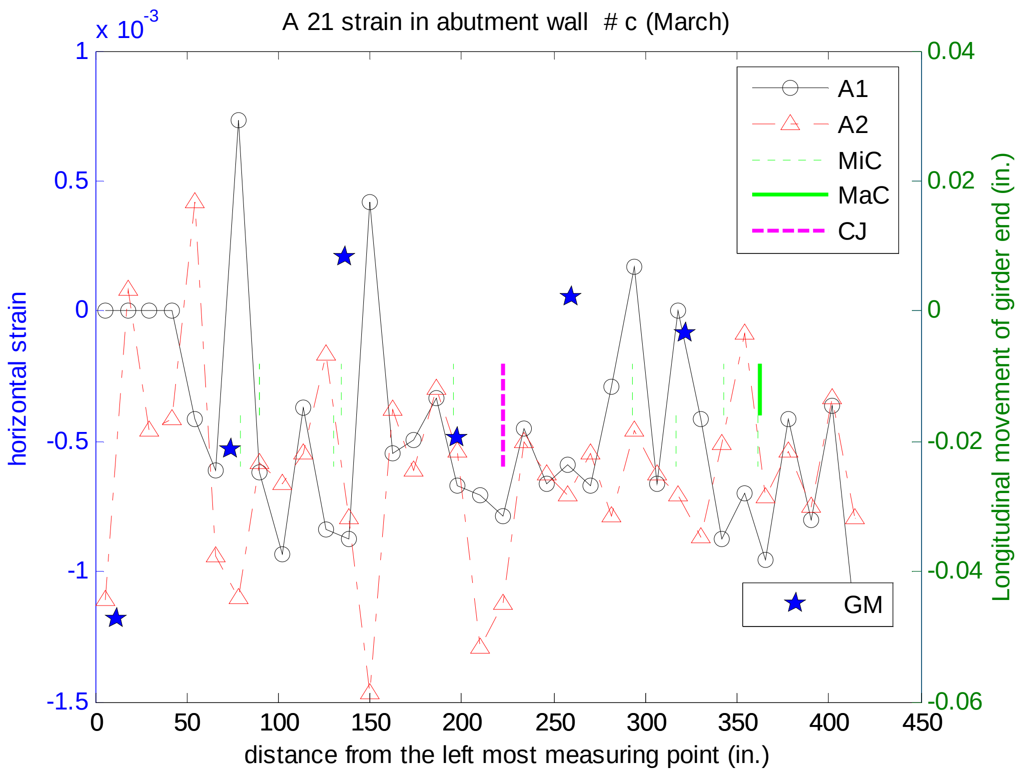


Figure C-14 Distribution of horizontal strain in abutment wall of Bridge A 2.1 in March 2007

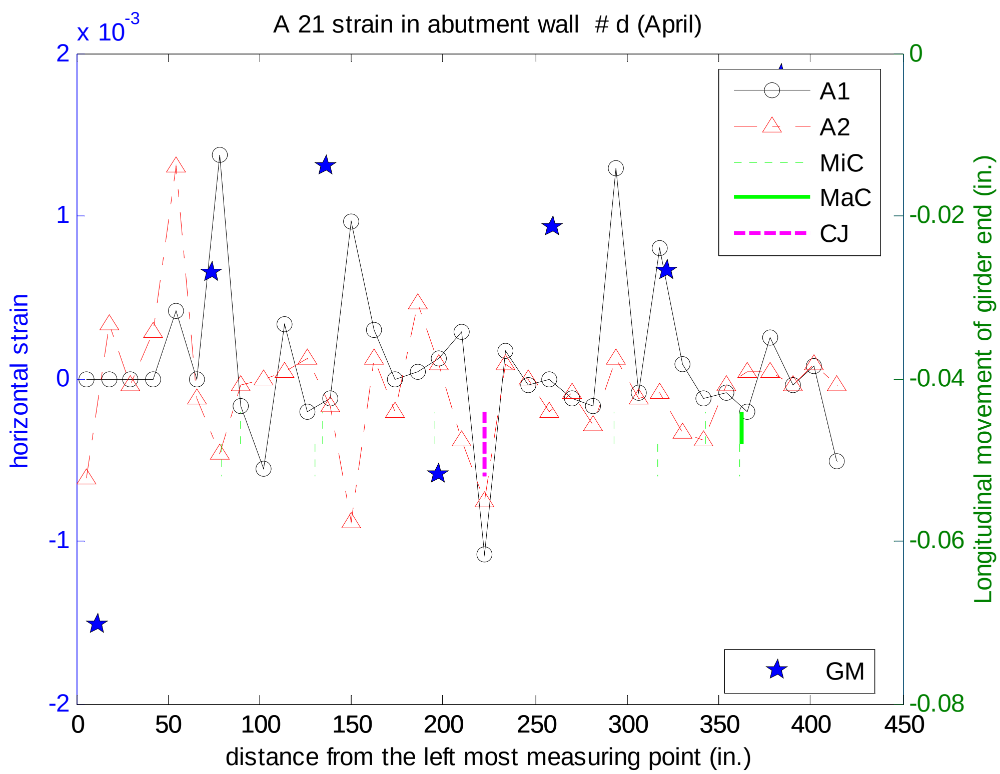


Figure C-15 Distribution of horizontal strain in abutment wall of Bridge A 2.1 in April 2007

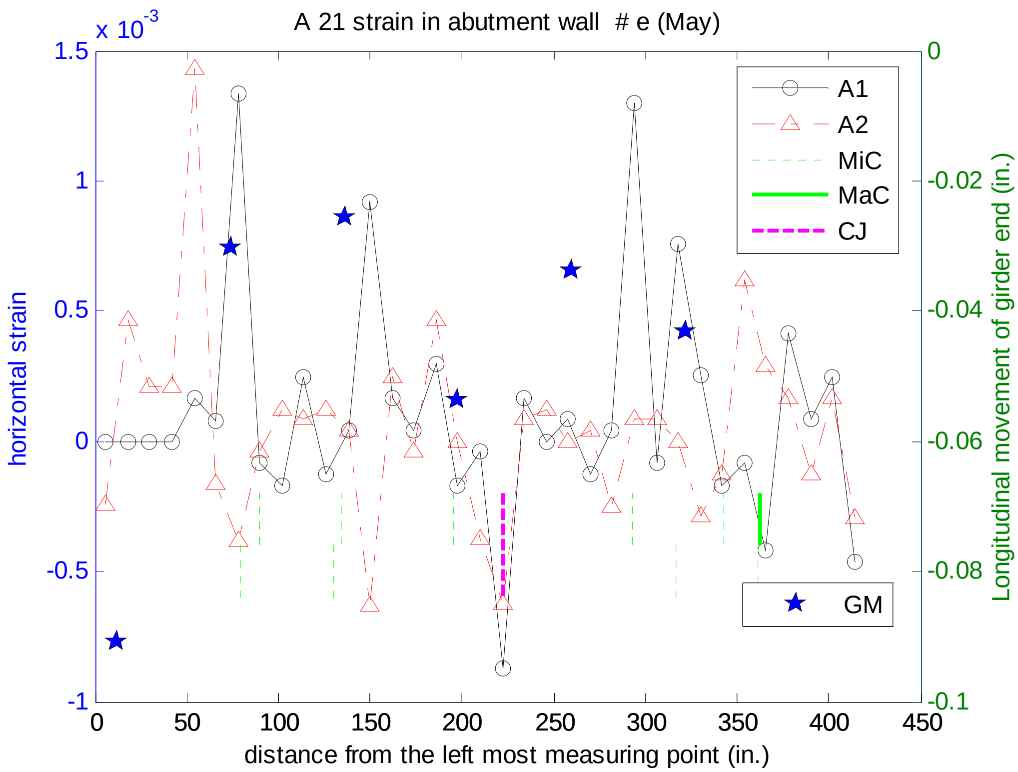


Figure C-16 Distribution of horizontal strain in abutment wall of Bridge A 2.1 in May 2007

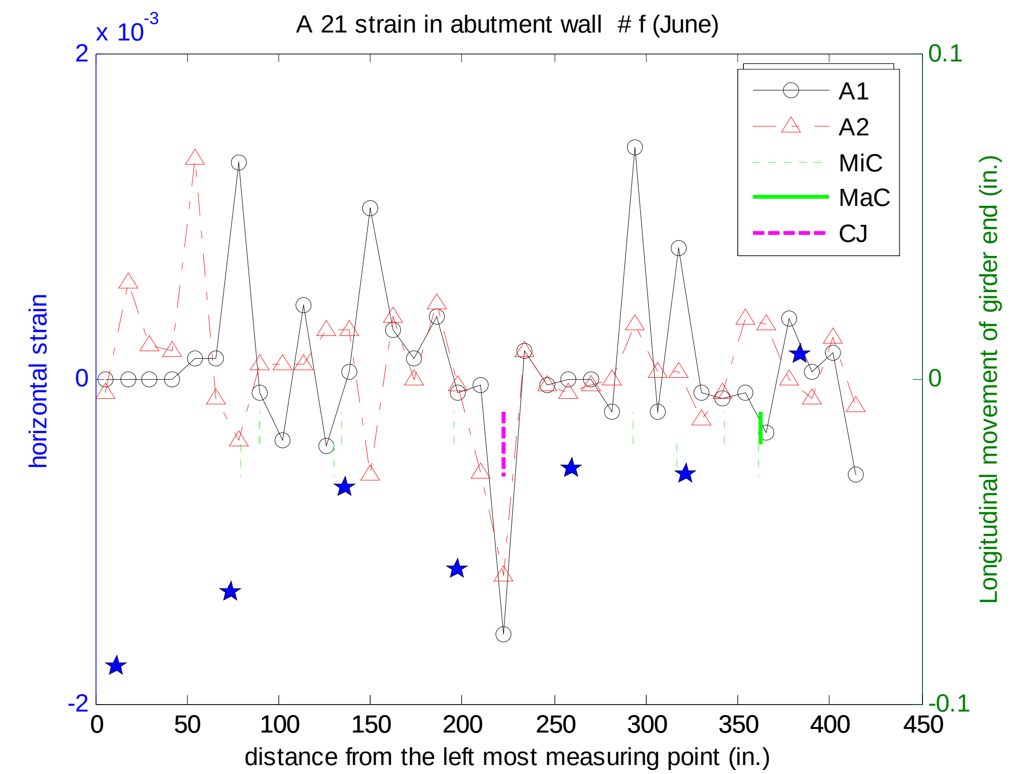


Figure C-17 Distribution of horizontal strain in abutment wall of Bridge A 2.1 in June 2007

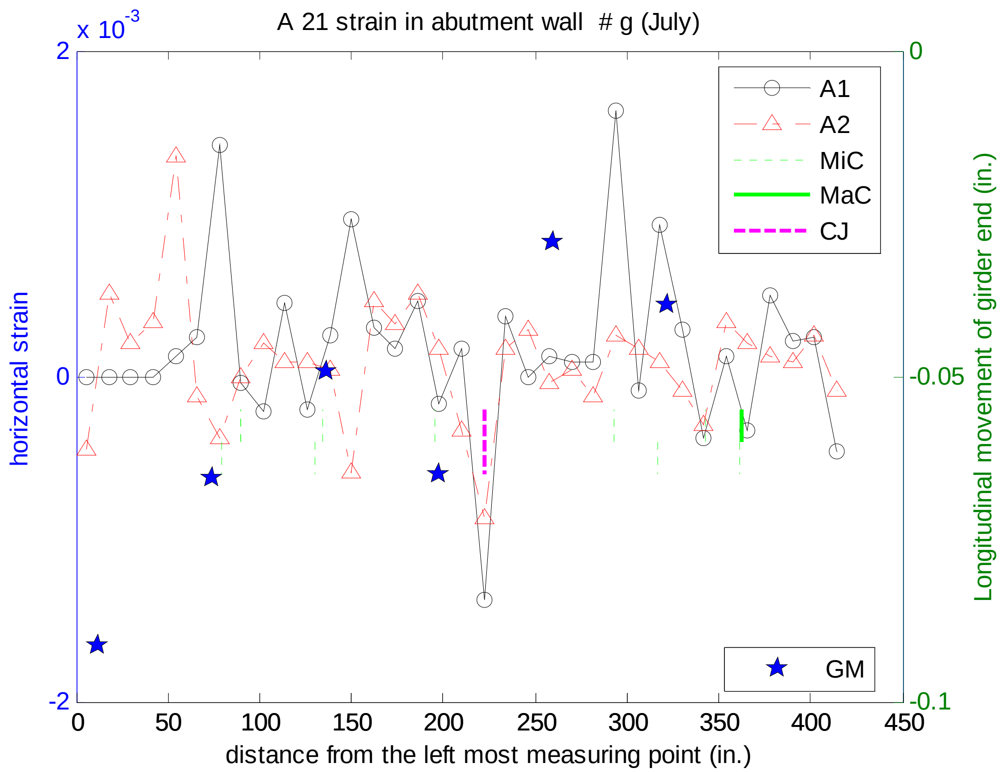


Figure C-18 Distribution of horizontal strain in abutment wall of Bridge A 2.1 in July 2007

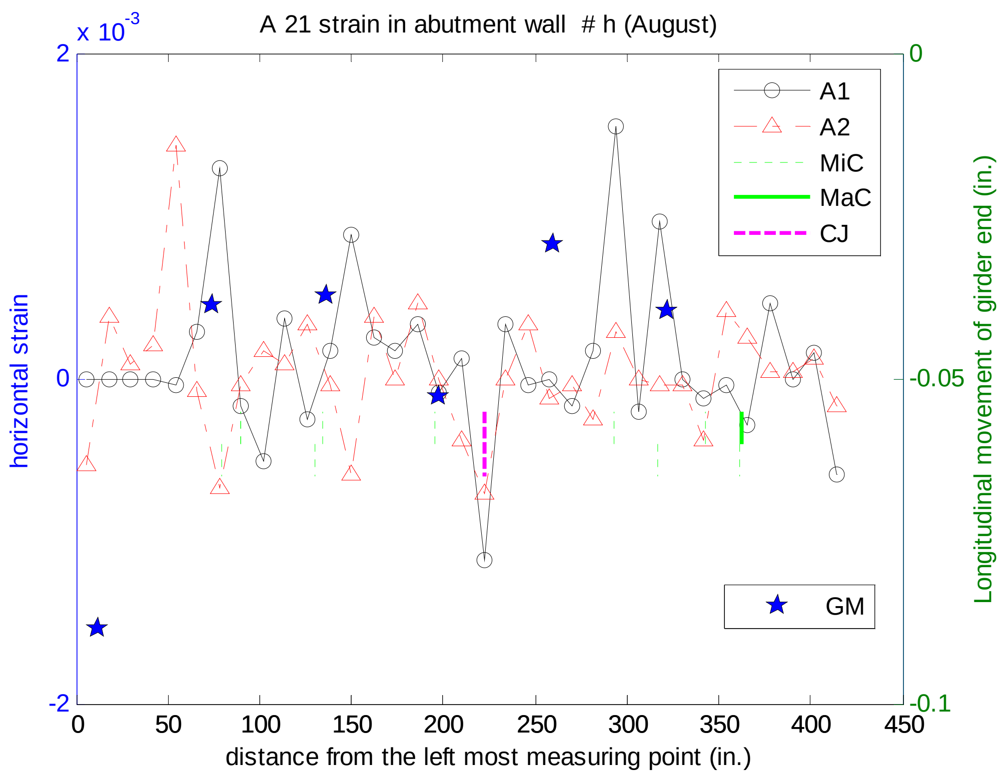


Figure C-19 Distribution of horizontal strain in abutment wall of Bridge A 2.1 in August 2007

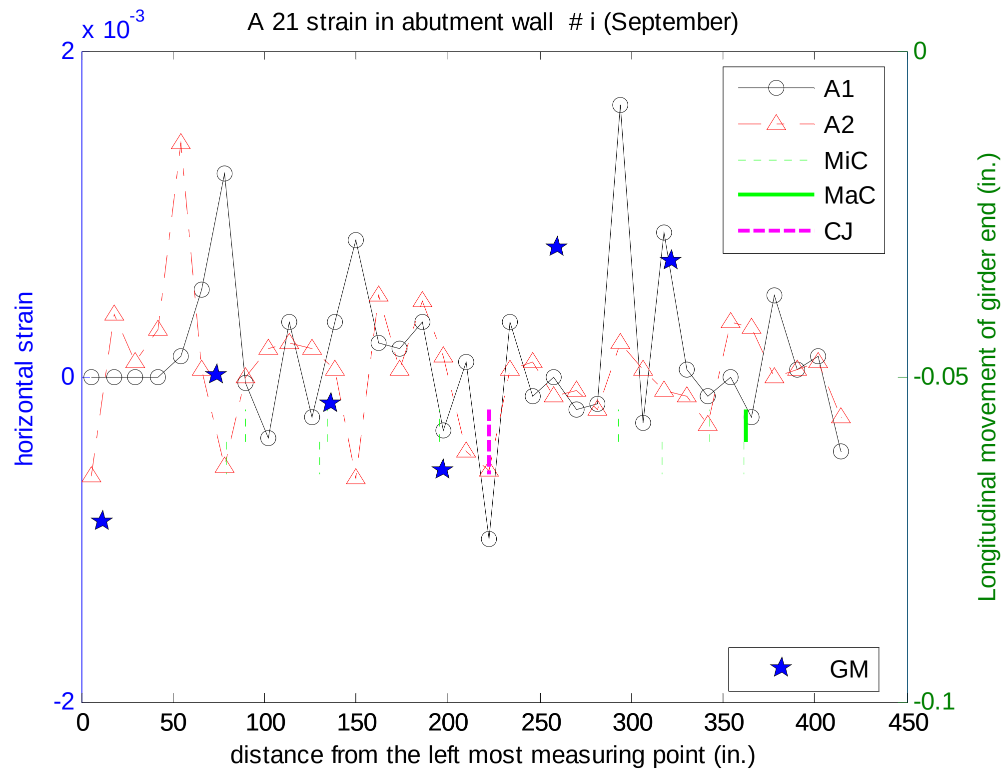


Figure C-20 Distribution of horizontal strain in abutment wall of Bridge A 2.1 in September 2007

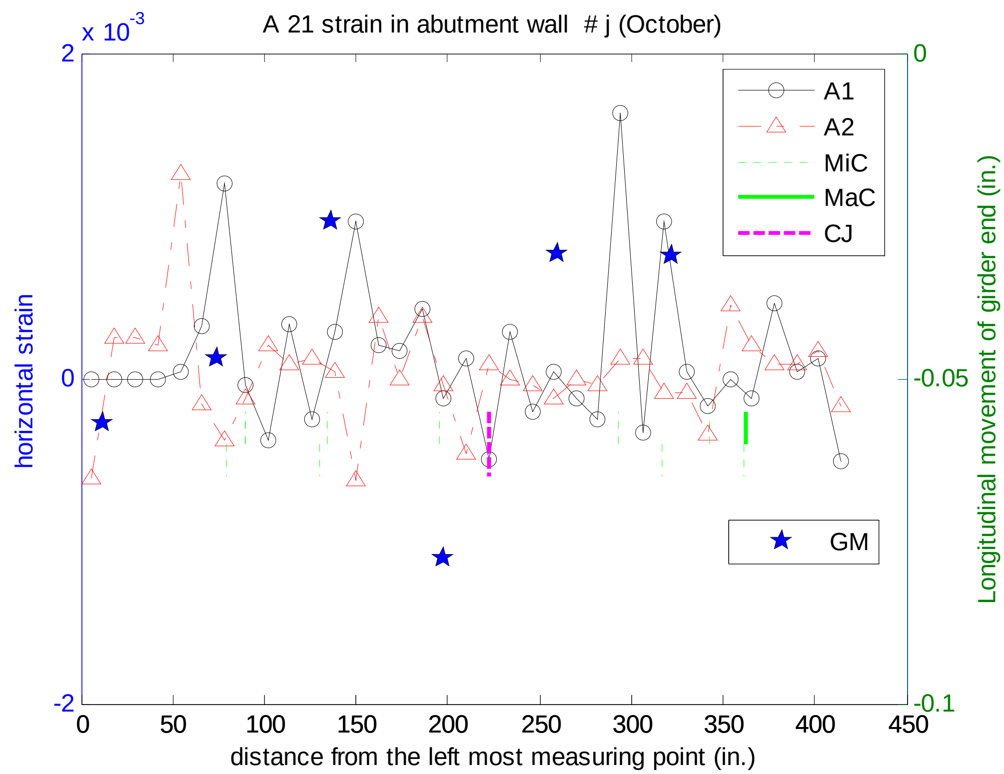


Figure C-21 Distribution of horizontal strain in abutment wall of Bridge A 2.1 in October 2007

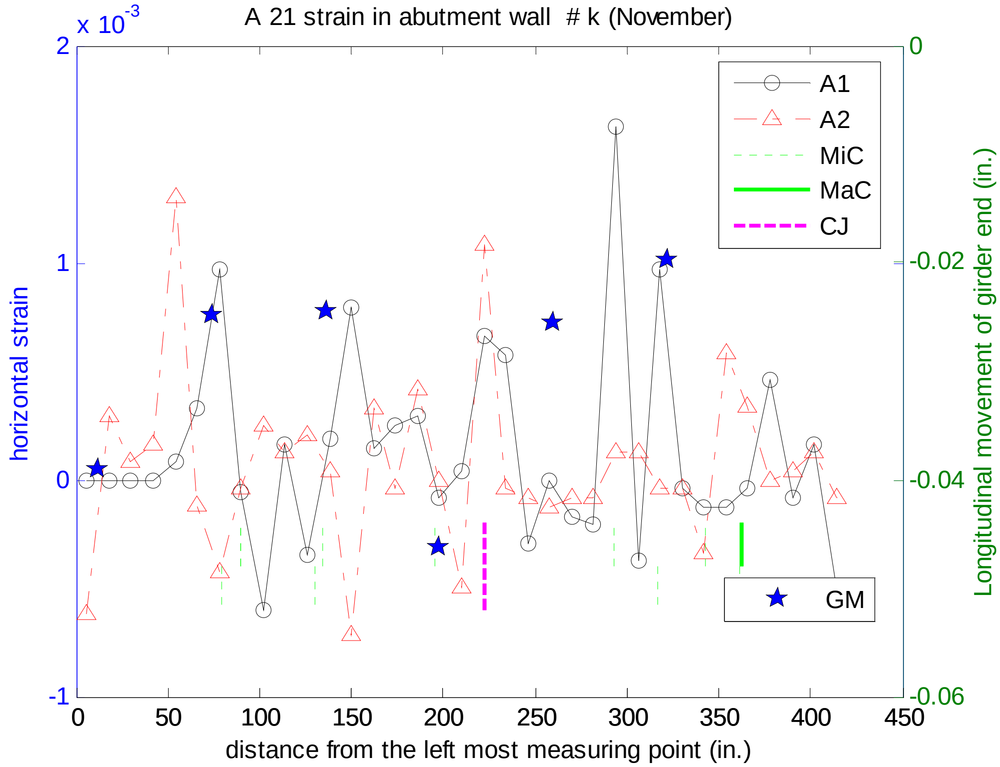


Figure C-22 Distribution of horizontal strain in abutment wall of Bridge A 2.1 in November 2007

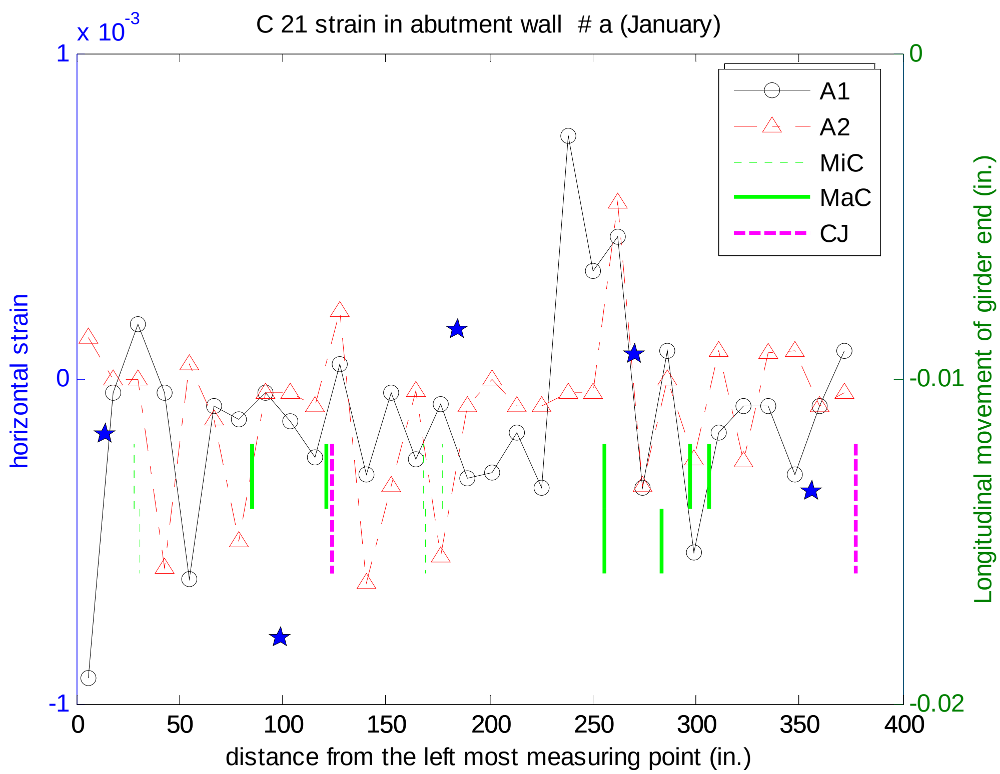


Figure C-23 Distribution of horizontal strain in abutment wall of Bridge C 2.1 in January 2007

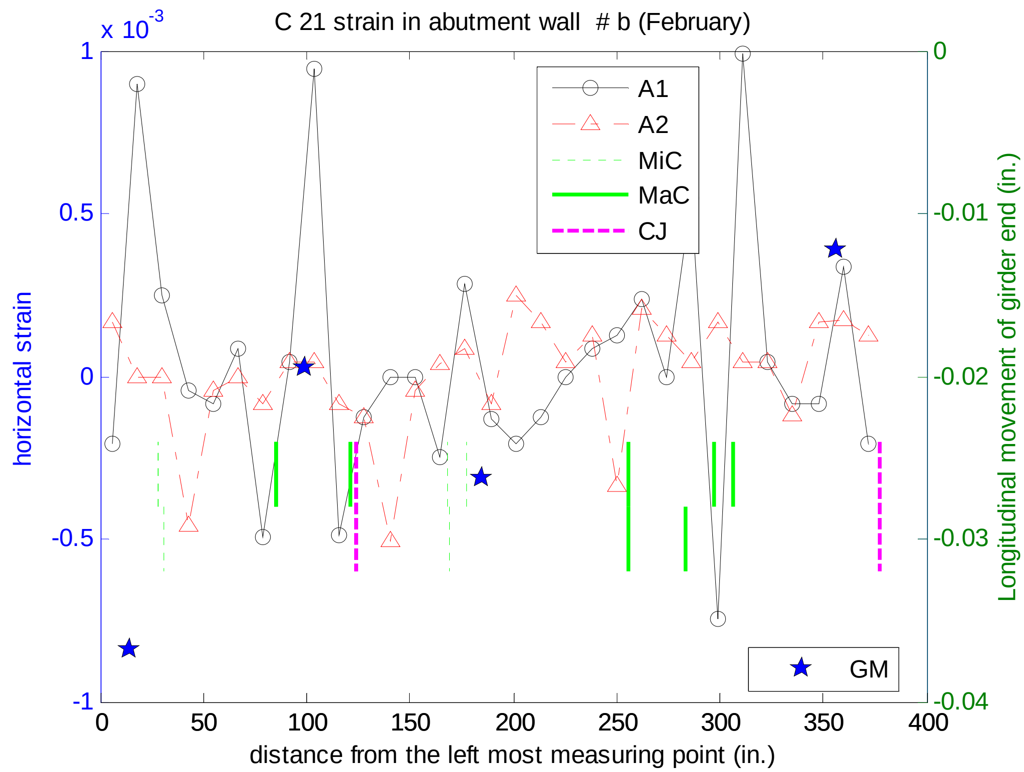


Figure C-24 Distribution of horizontal strain in abutment wall of Bridge C 2.1 in February 2007

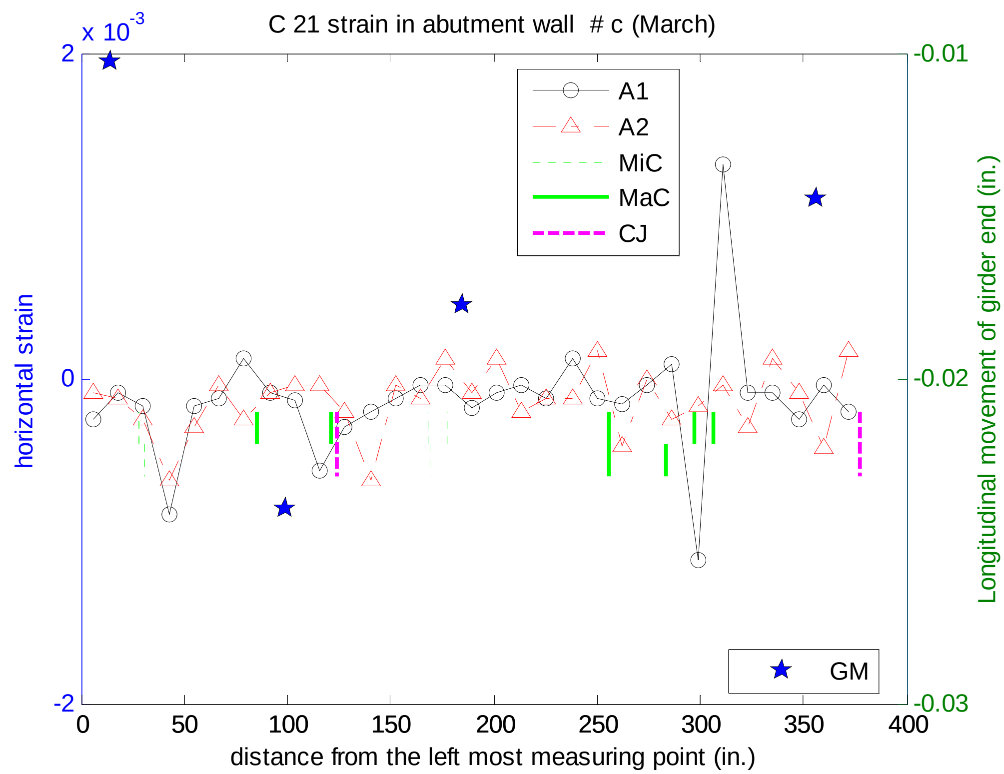


Figure C-25 Distribution of horizontal strain in abutment wall of Bridge C 2.1 in March 2007

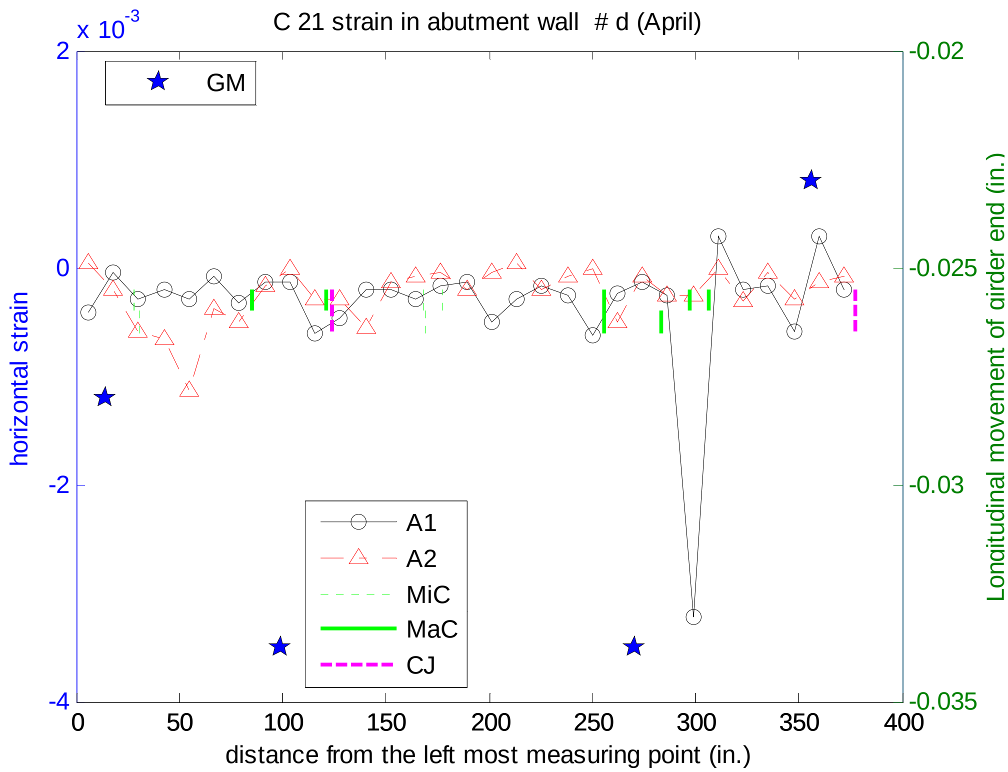


Figure C-26 Distribution of horizontal strain in abutment wall of Bridge C 2.1 in April 2007

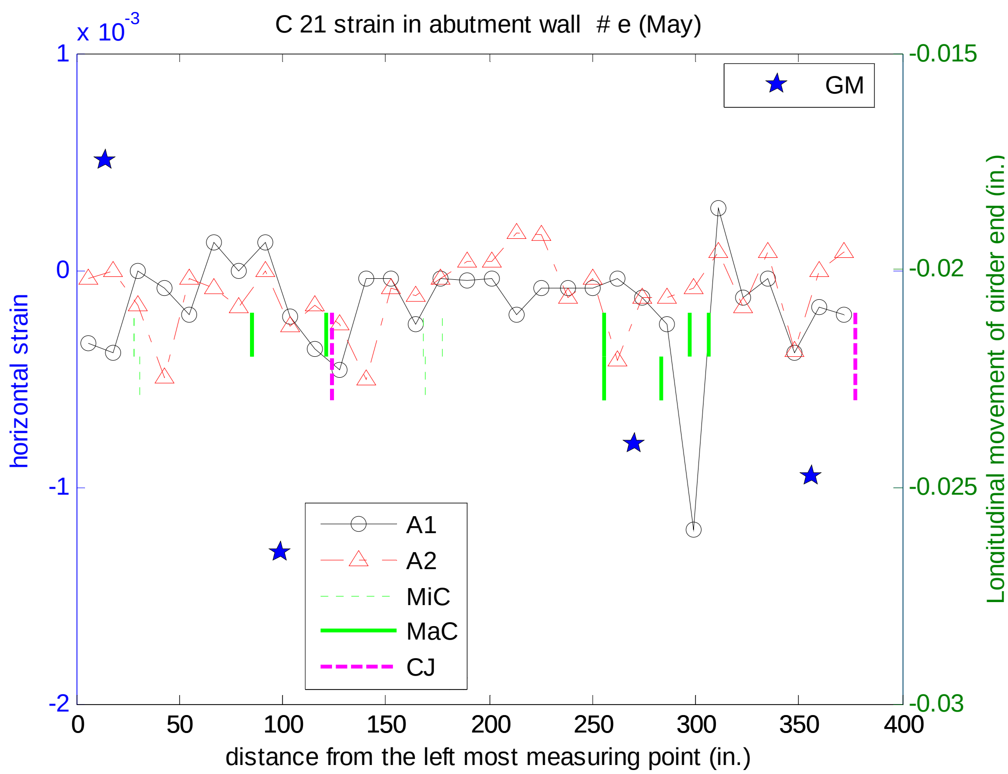


Figure C-27 Distribution of horizontal strain in abutment wall of Bridge C 2.1 in May 2007

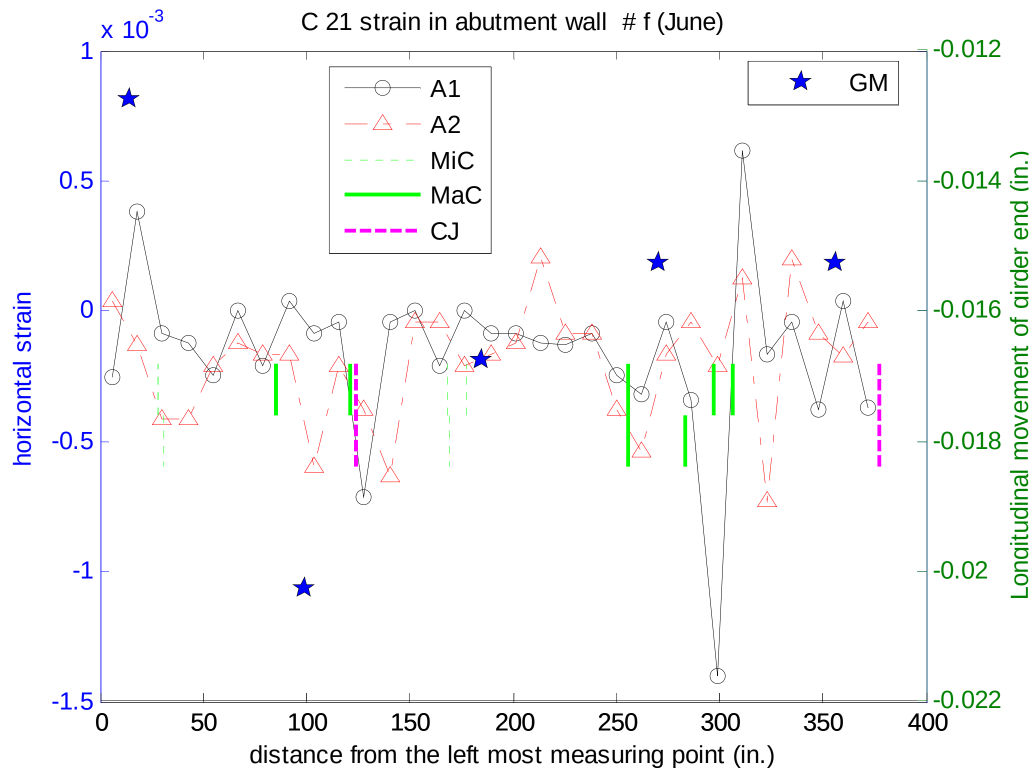


Figure C-28 Distribution of horizontal strain in abutment wall of Bridge C 2.1 in June 2007

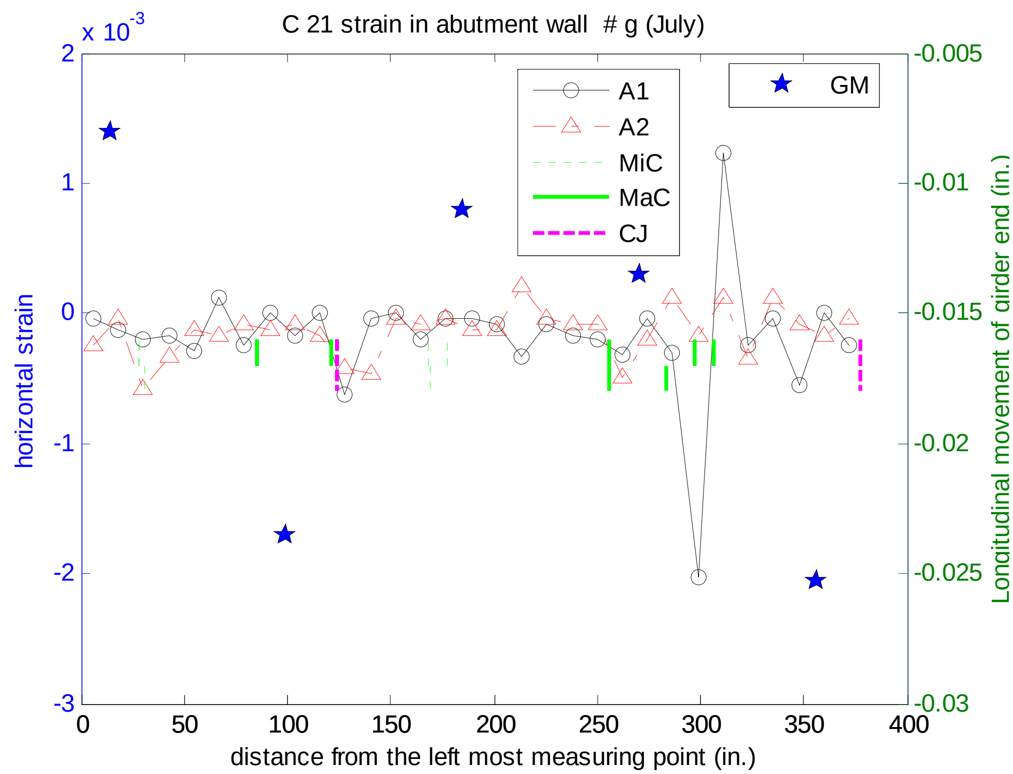


Figure C-29 Distribution of horizontal strain in abutment wall of Bridge C 2.1 in July 2007

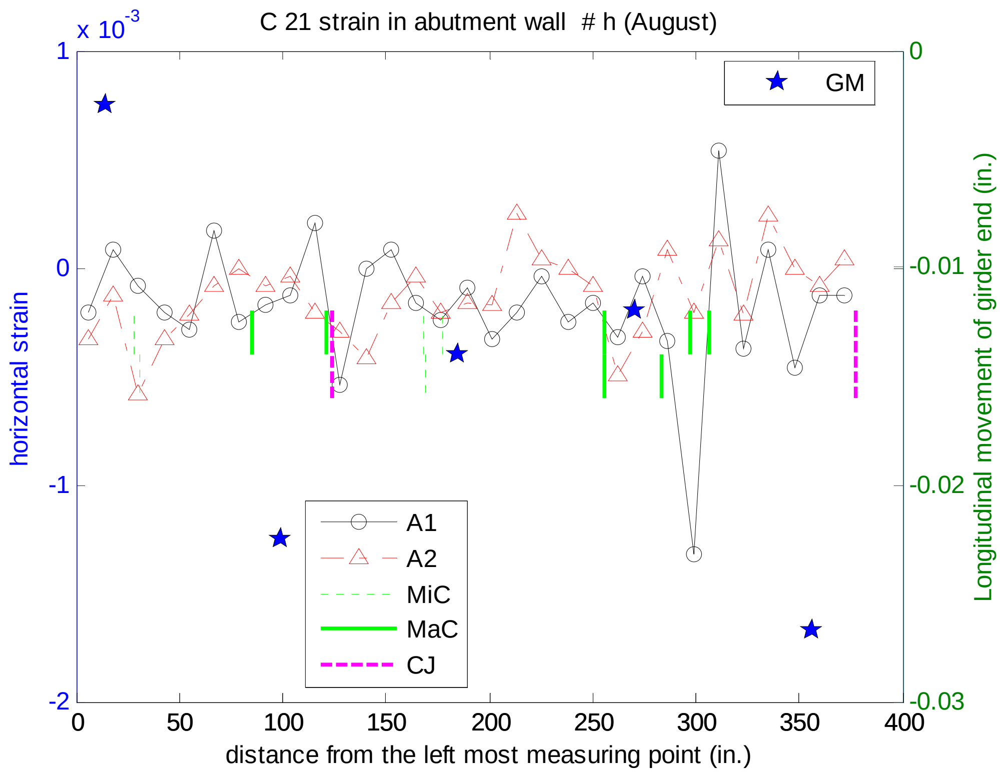


Figure C-30 Distribution of horizontal strain in abutment wall of Bridge C 2.1 in August 2007

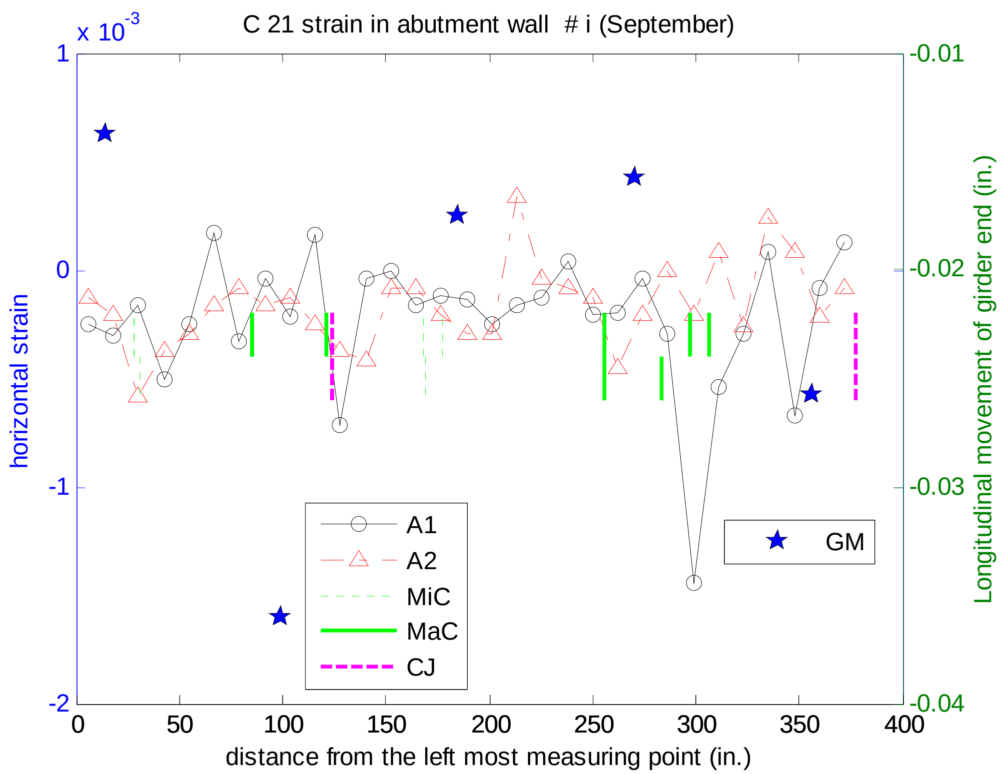


Figure C-31 Distribution of horizontal strain in abutment wall of Bridge C 2.1 in September 2007

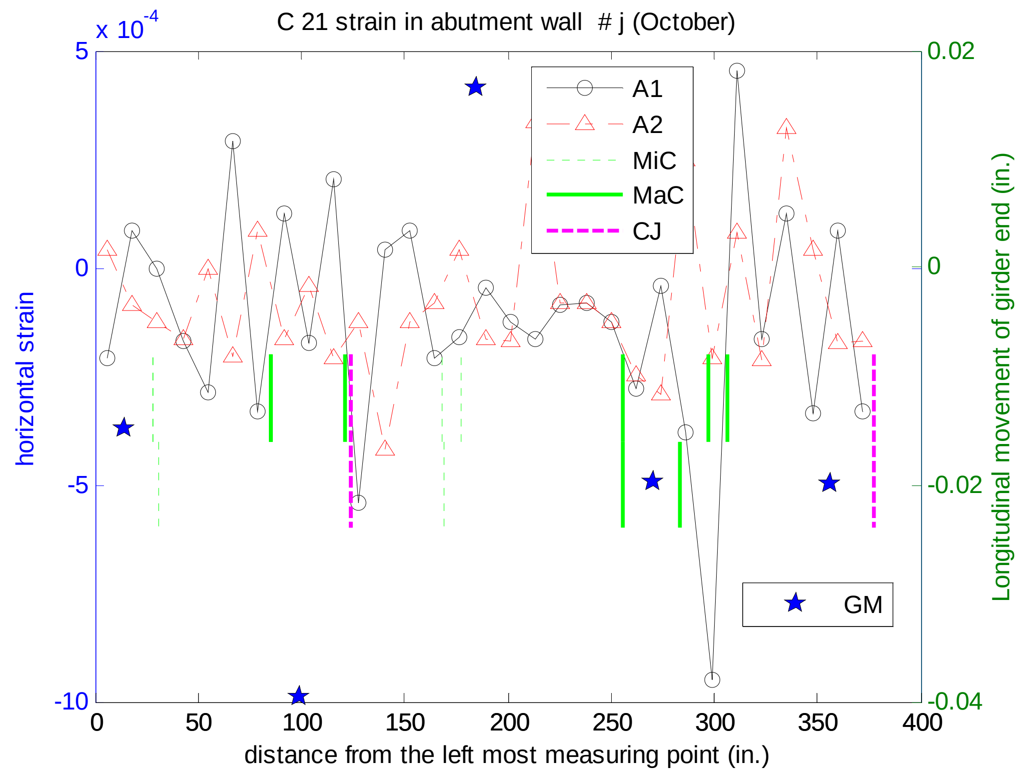


Figure C-32 Distribution of horizontal strain in abutment wall of Bridge C 2.1 in October 2007

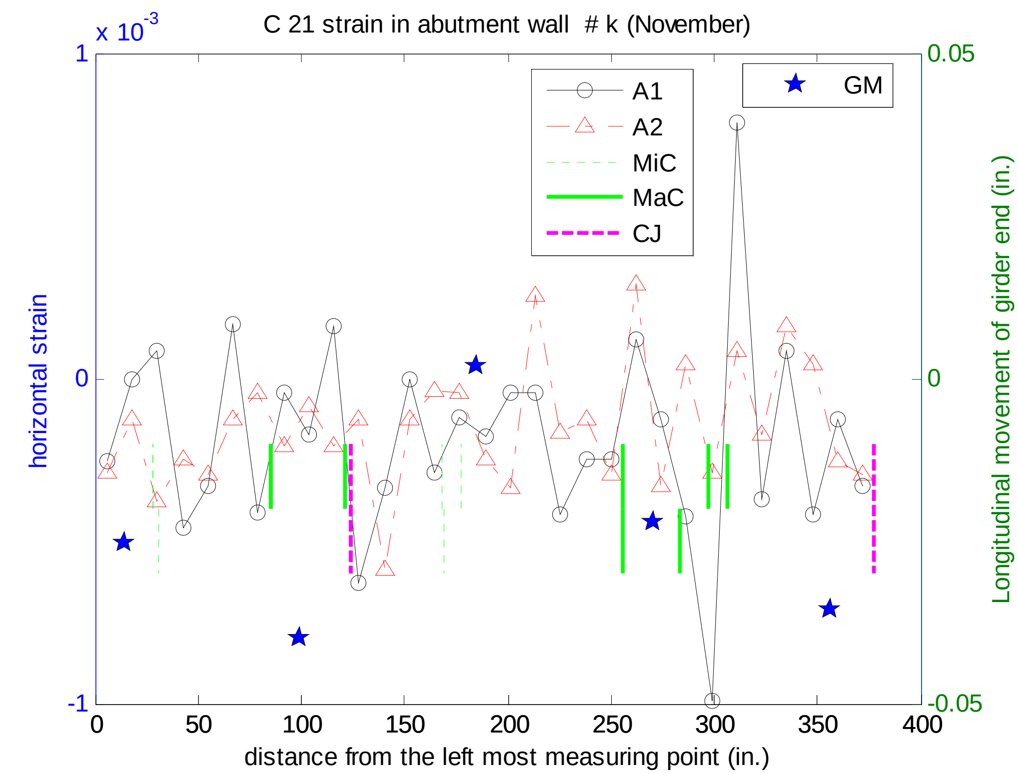


Figure C-33 Distribution of horizontal strain in abutment wall of Bridge C 2.1 in November 2007

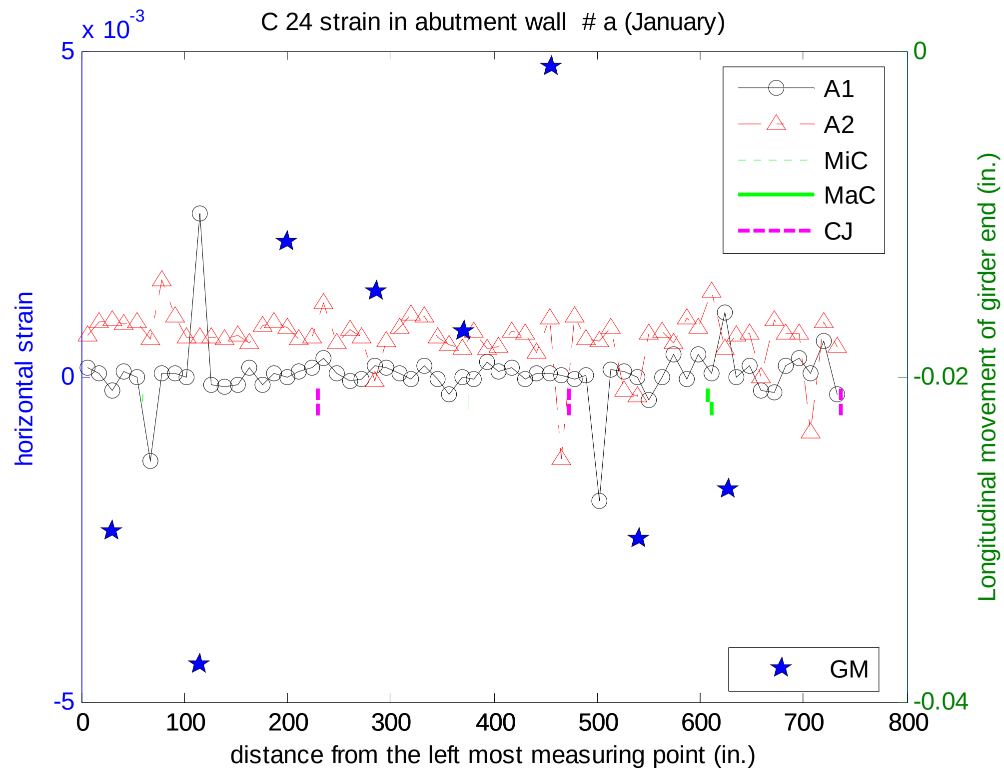


Figure C-34 Distribution of horizontal strain in abutment wall of Bridge C 2.1 in January 2007

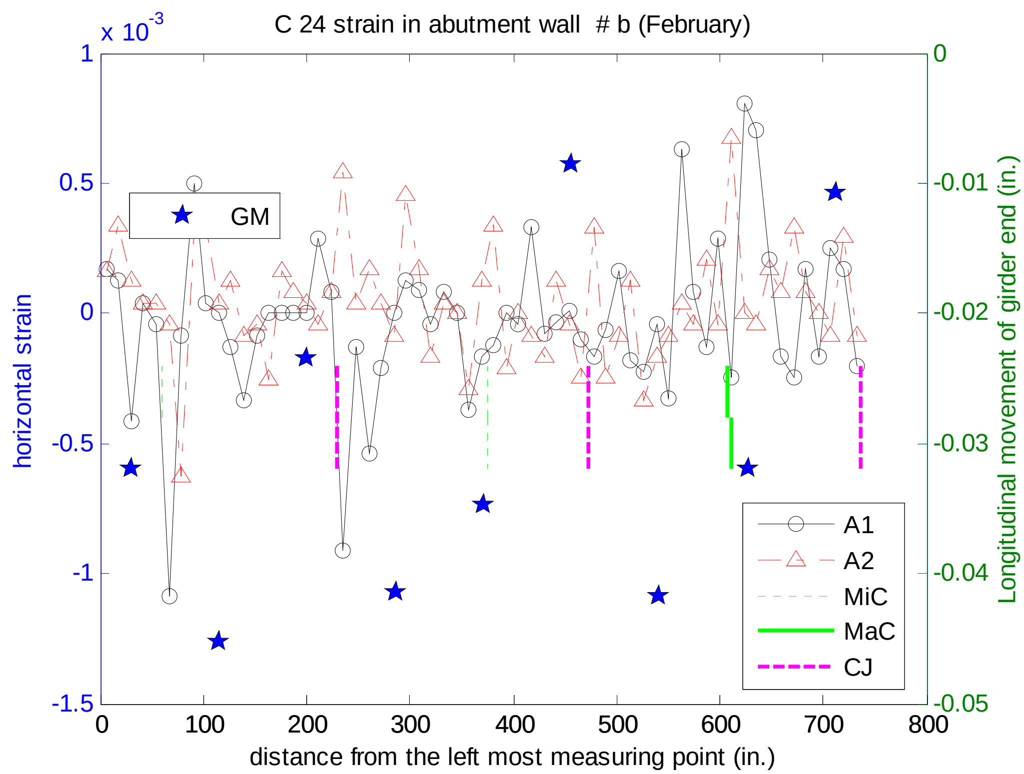


Figure C-35 Distribution of horizontal strain in abutment wall of Bridge C 2.4 in March 2007

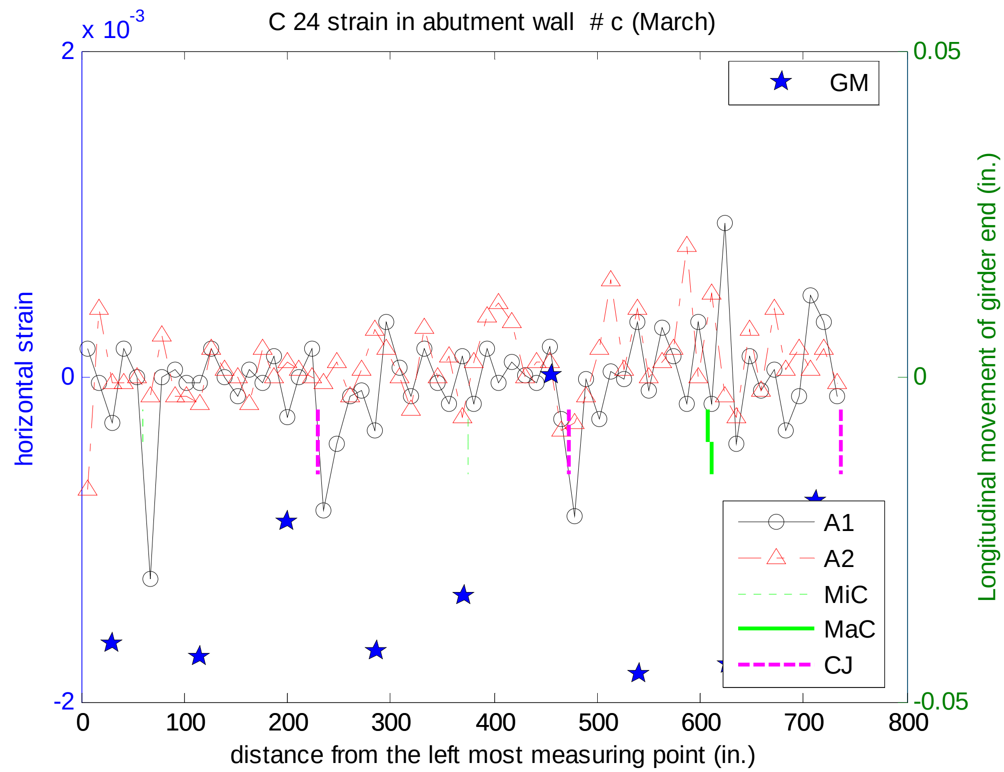


Figure C-36 Distribution of horizontal strain in abutment wall of Bridge C 2.4 in March 2007

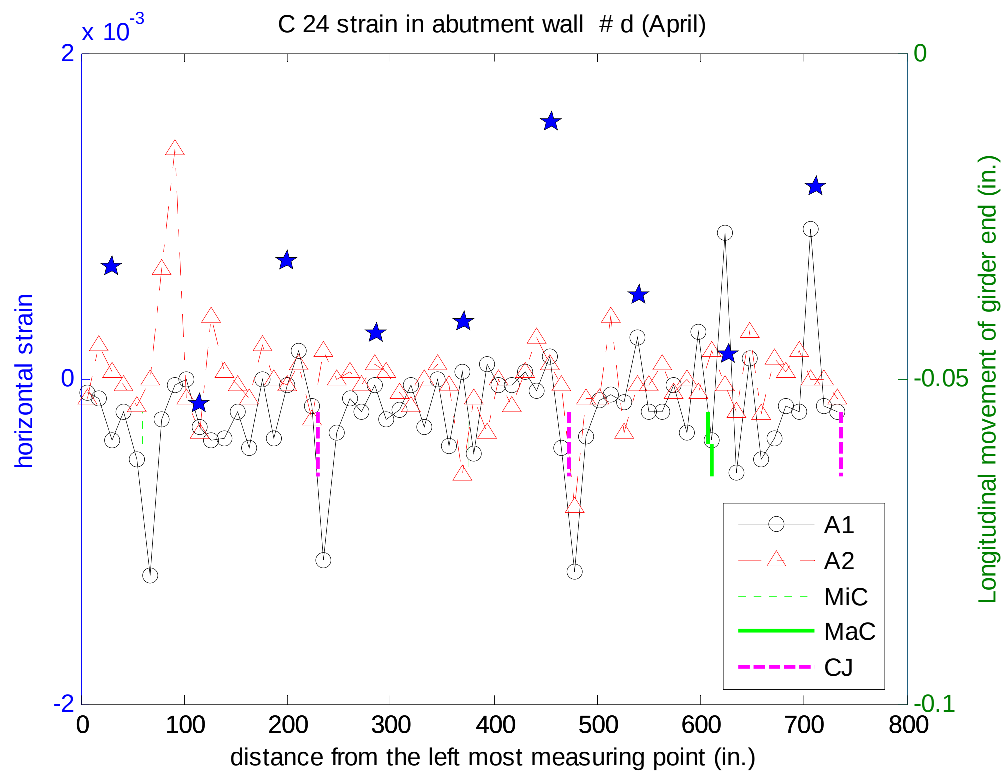


Figure C-37 Distribution of horizontal strain in abutment wall of Bridge C 2.4 in April 2007

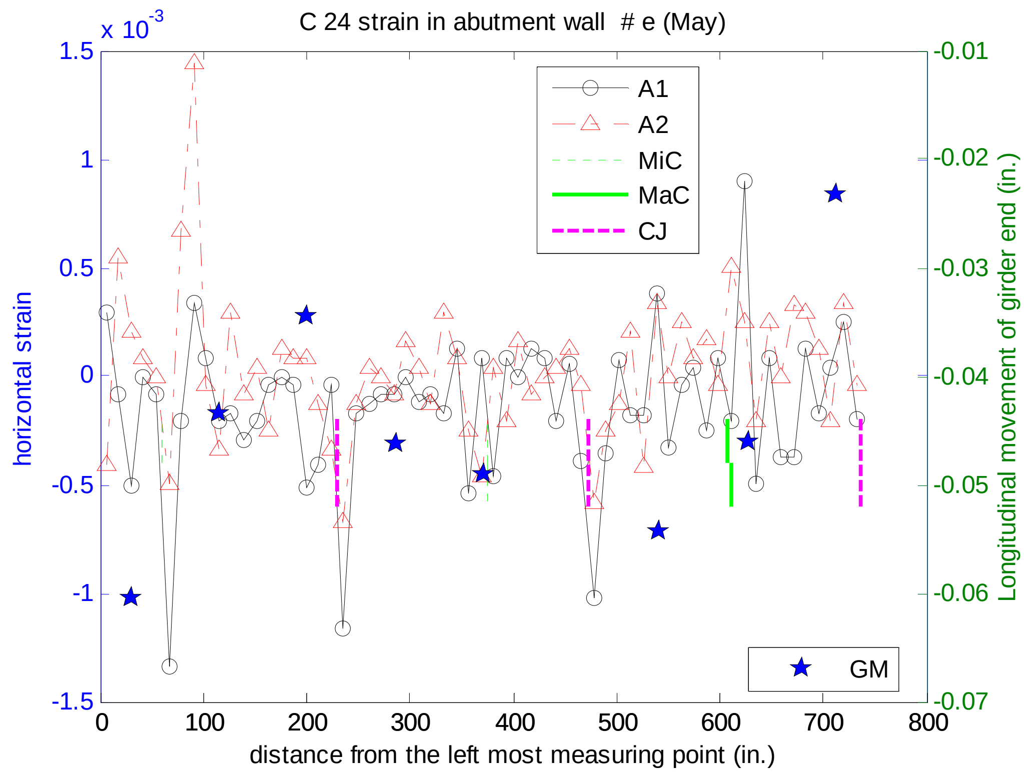


Figure C-38 Distribution of horizontal strain in abutment wall of Bridge C 2.4 in May 2007

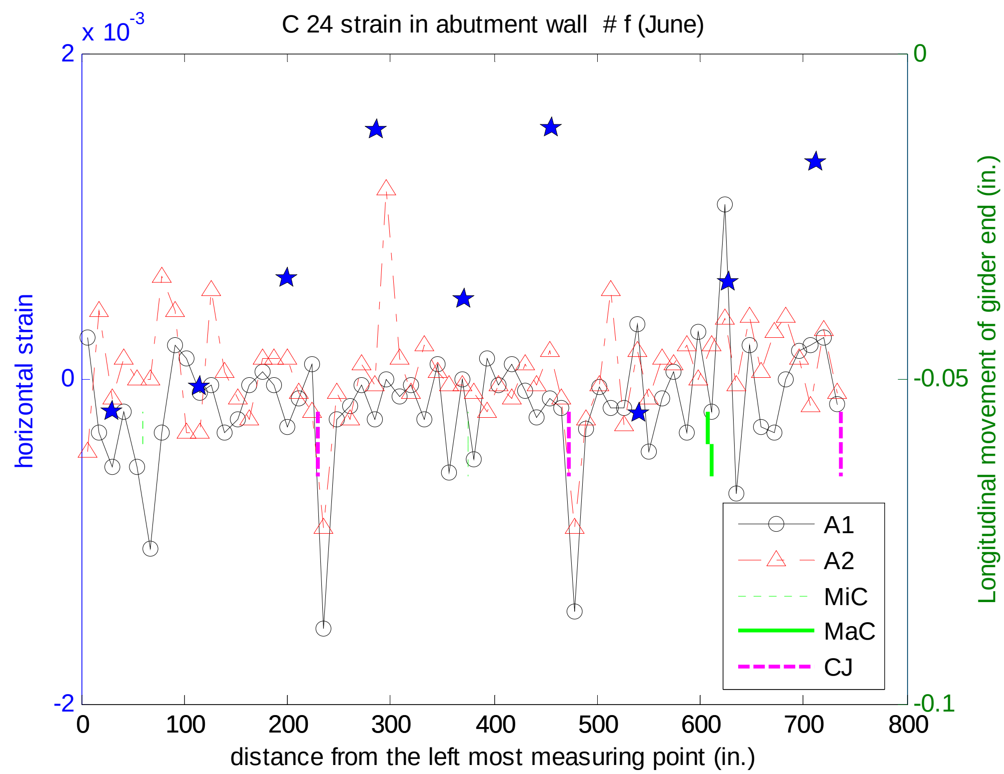


Figure C-39 Distribution of horizontal strain in abutment wall of Bridge C 2.4 in June 2007

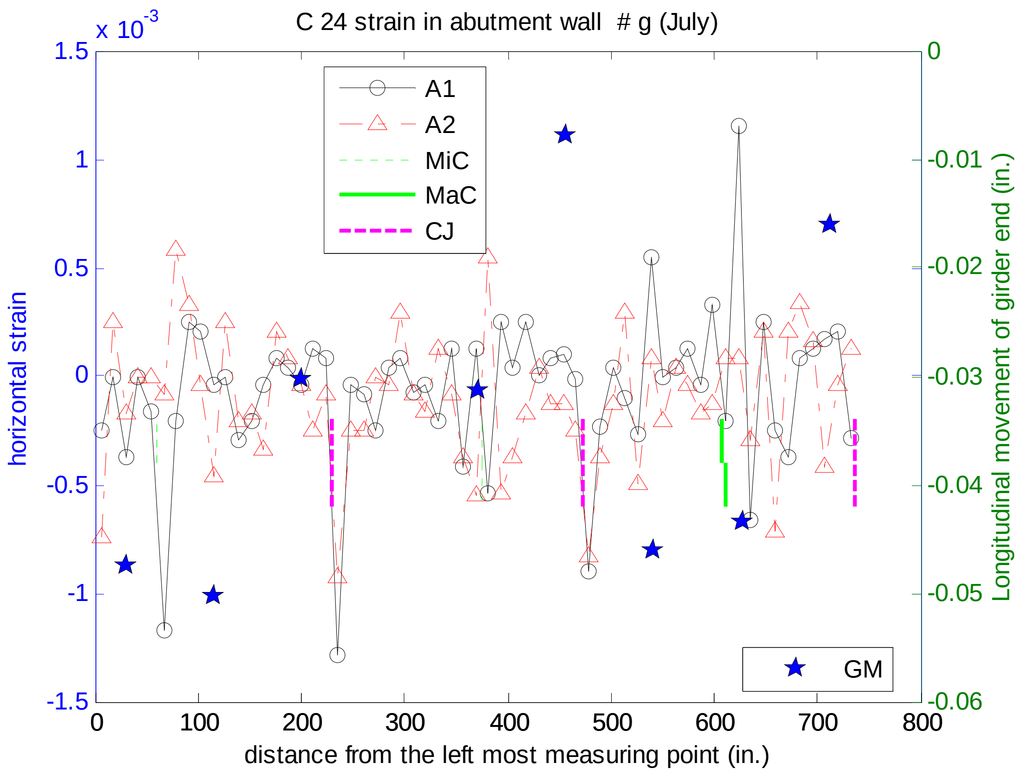


Figure C-40 Distribution of horizontal strain in abutment wall of Bridge C 2.4 in July 2007

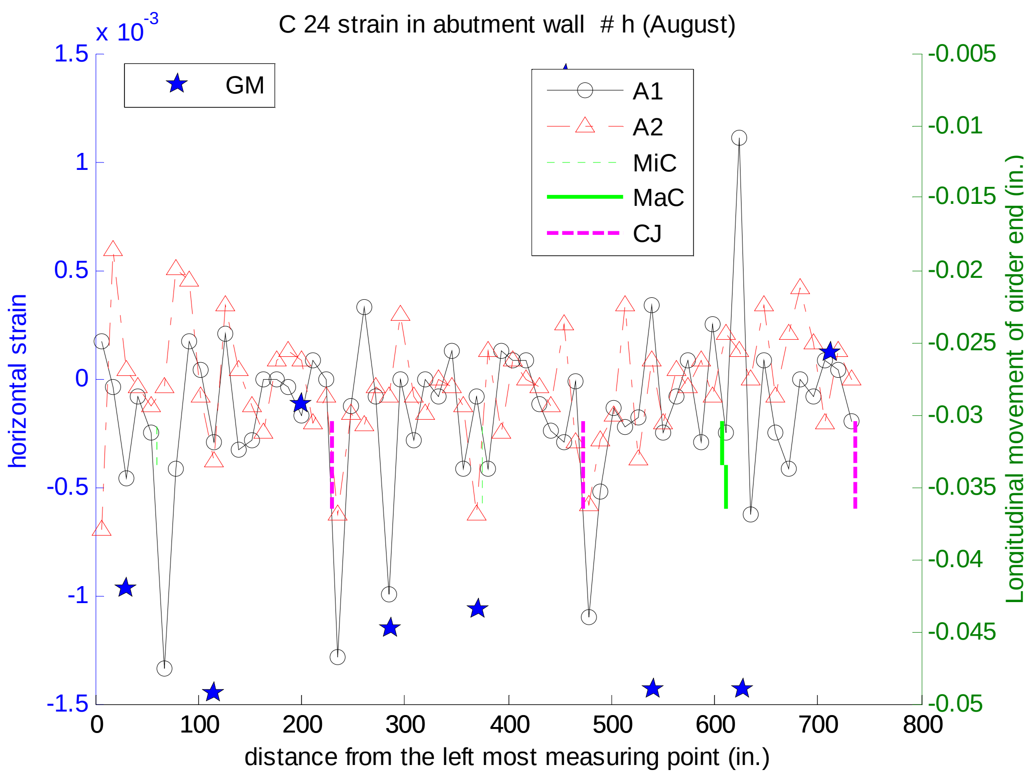


Figure C-41 Distribution of horizontal strain in abutment wall of Bridge C 2.4 in August 2007

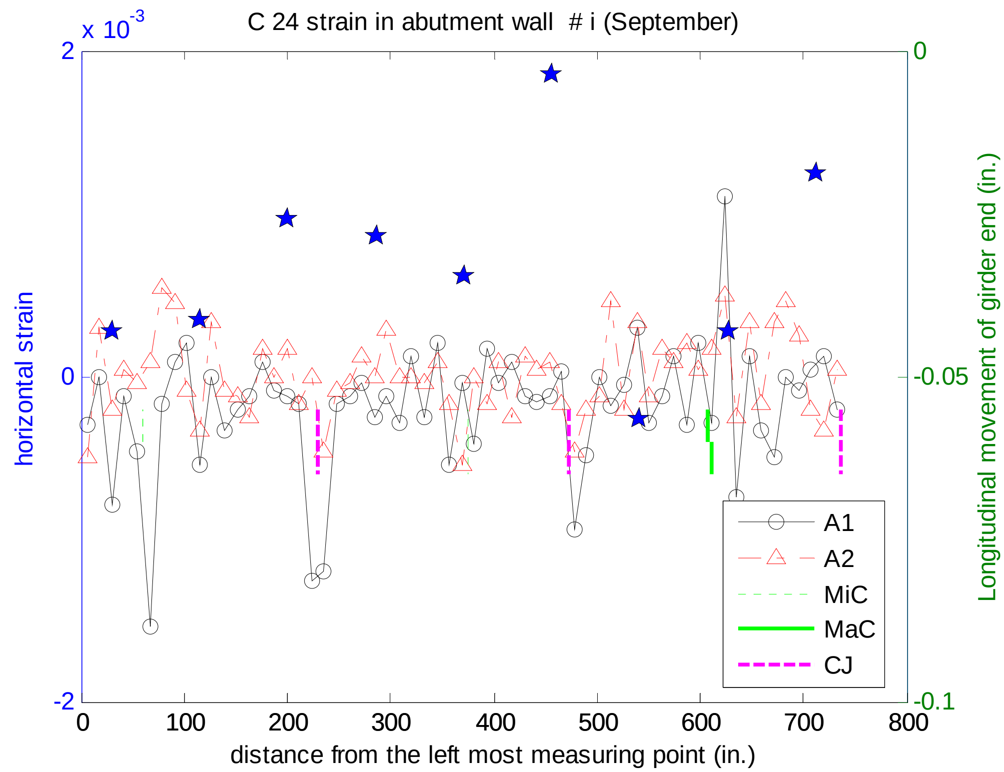


Figure C-42 Distribution of horizontal strain in abutment wall of Bridge C 2.4 in September 2007

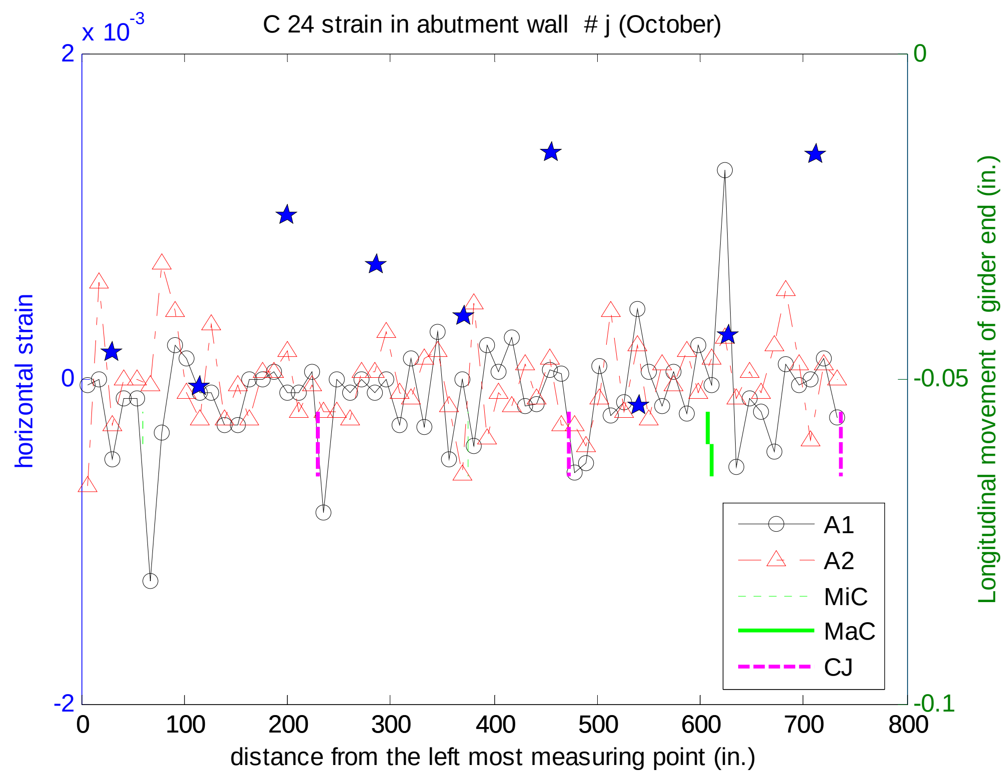


Figure C-43 Distribution of horizontal strain in abutment wall of Bridge C 2.4 in October 2007

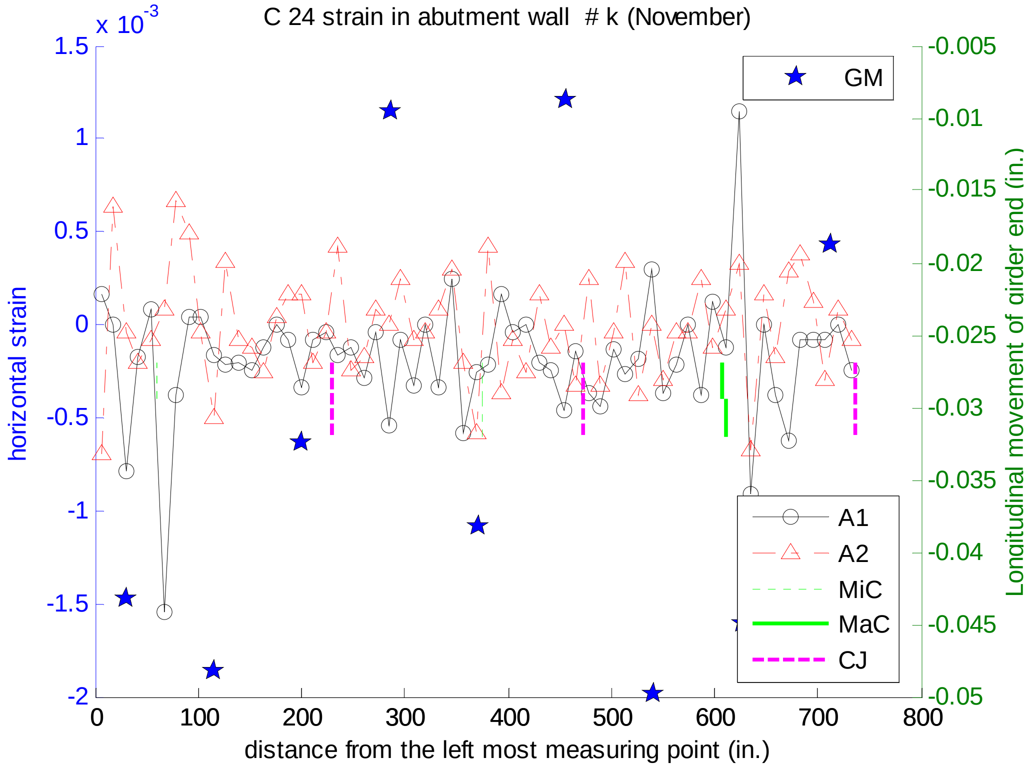


Figure C-44 Distribution of horizontal strain in abutment wall of Bridge C 2.4 in November 2007

C.I.2 Distribution of strains along abutment walls

The distributions of horizontal strains in the backwall of bridge A 1.7 are shown in Figure C-45 to **Figure** C-55. Similarly, the distributions of horizontal strains in the backwalls of bridges A 2.1, C 2.1, and C 2.4 are shown in **Figure** C-56 to **Figure** C-66, **Figure** C-67 to **Figure** C-77, and **Figure** C-78 to **Figure** C-88; respectively.

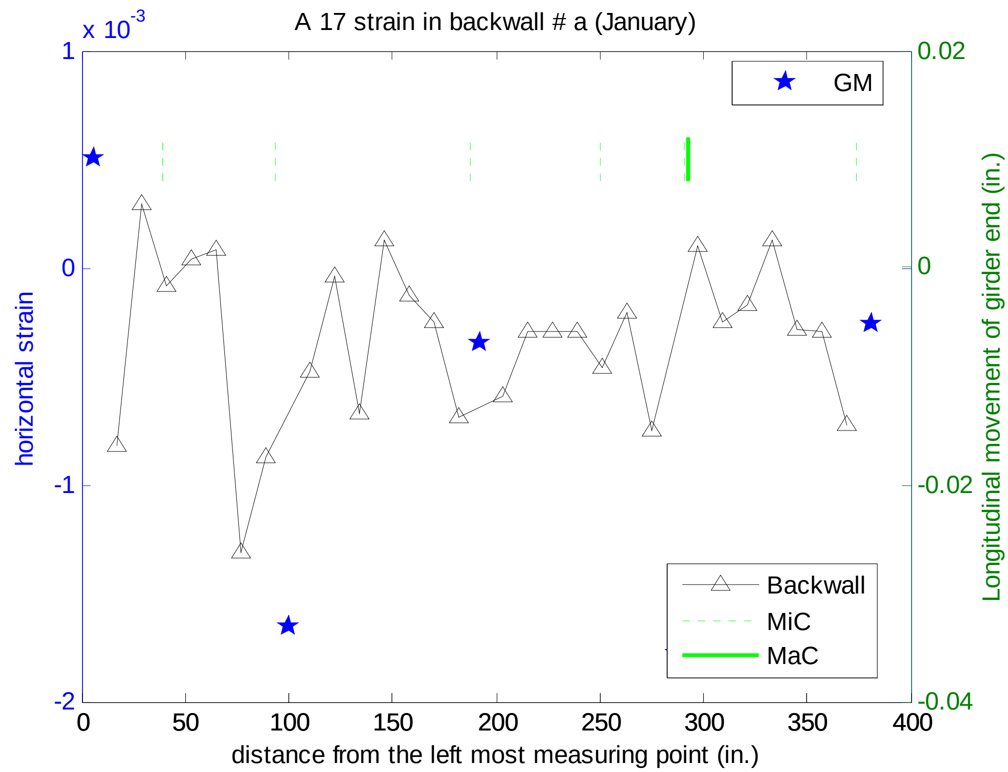


Figure C-45 Distribution of horizontal strain in backwall of Bridge A 1.7 in January 2007

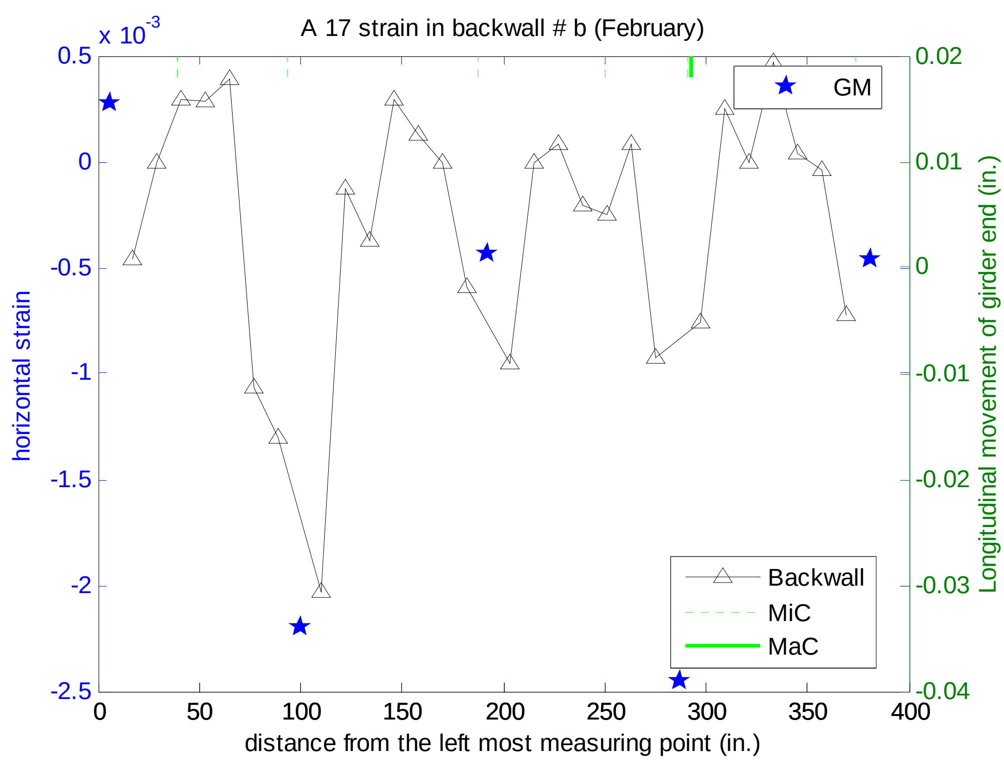


Figure C-46 Distribution of horizontal strain in backwall of Bridge A 1.7 in February 2007

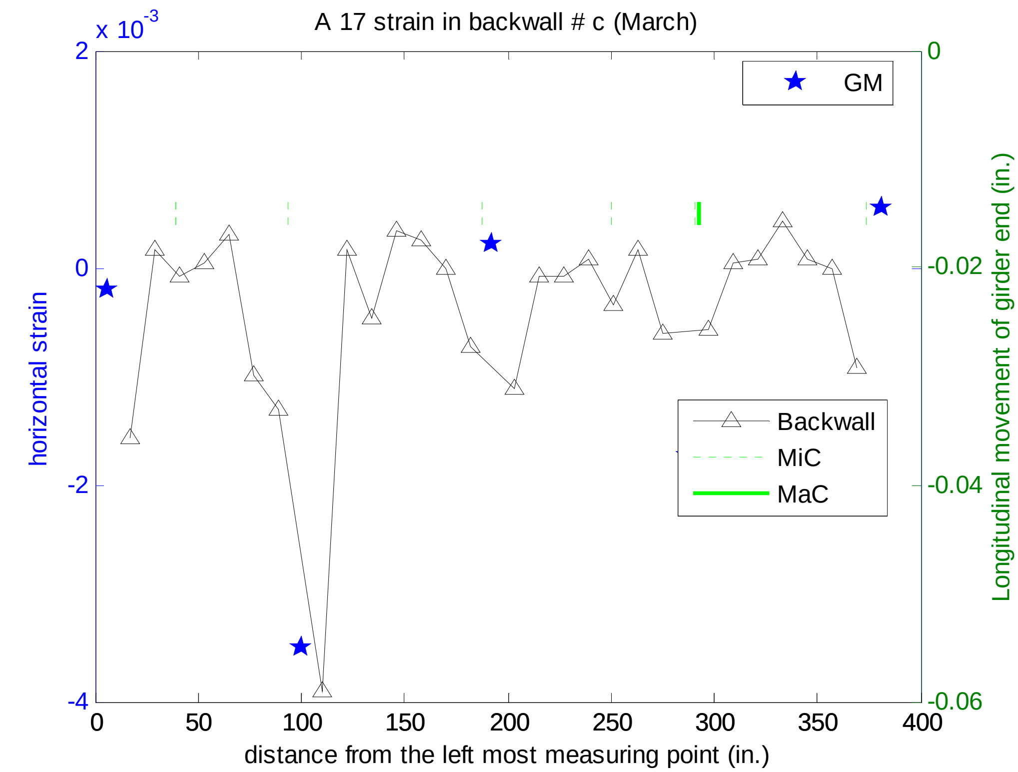


Figure C-47 Distribution of horizontal strain in backwall of Bridge A 1.7 in March 2007

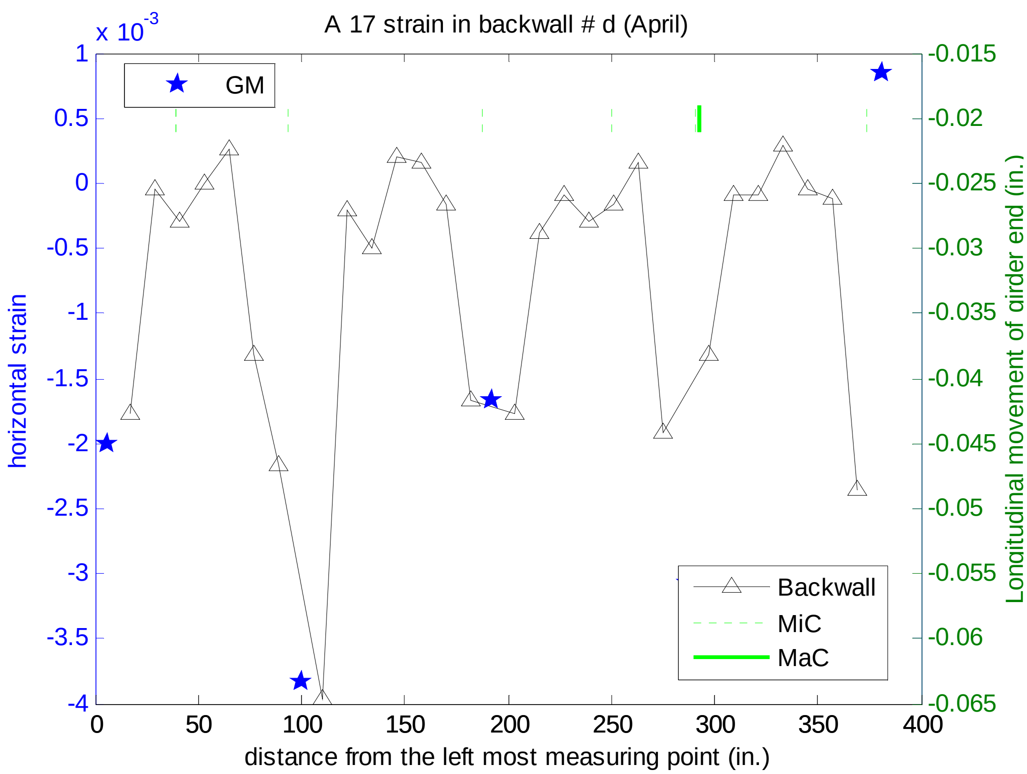


Figure C-48 Distribution of horizontal strain in backwall of Bridge A 1.7 in April 2007

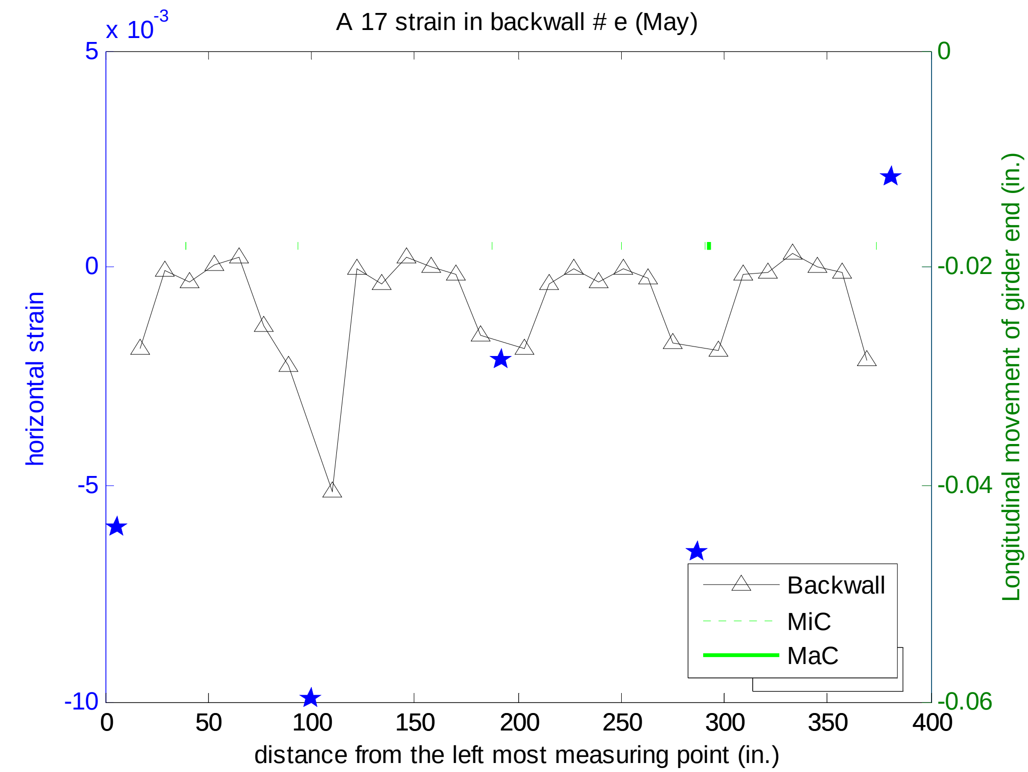


Figure C-49 Distribution of horizontal strain in backwall of Bridge A 1.7 in May 2007

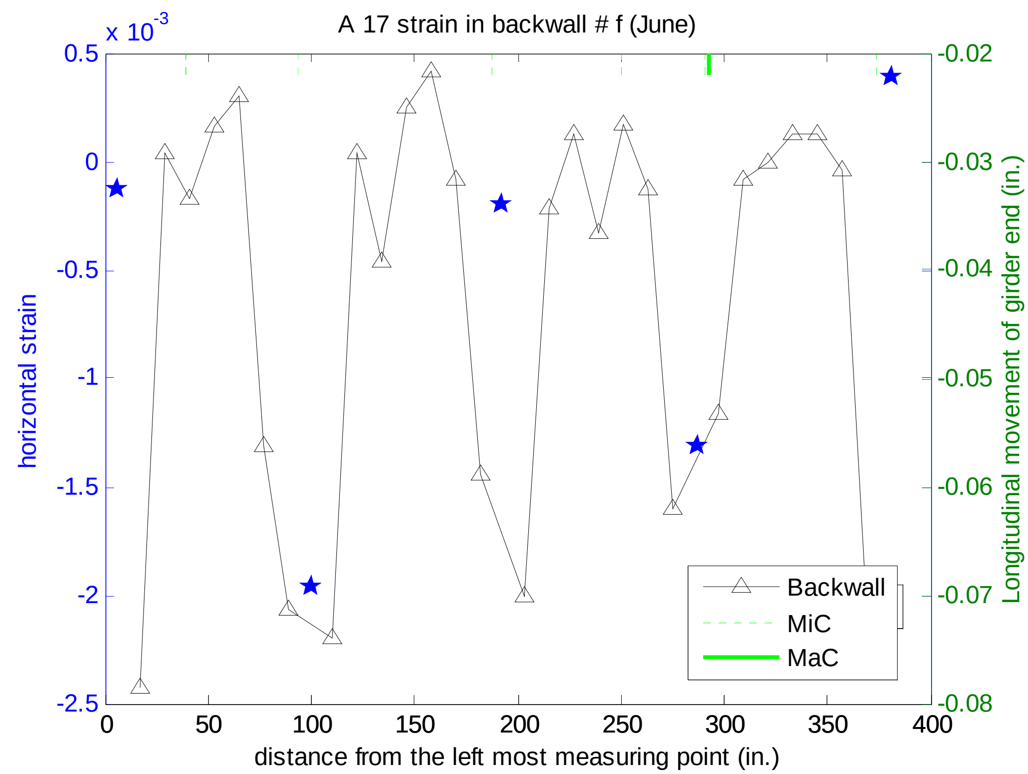


Figure C-50 Distribution of horizontal strain in backwall of Bridge A 1.7 in June 2007

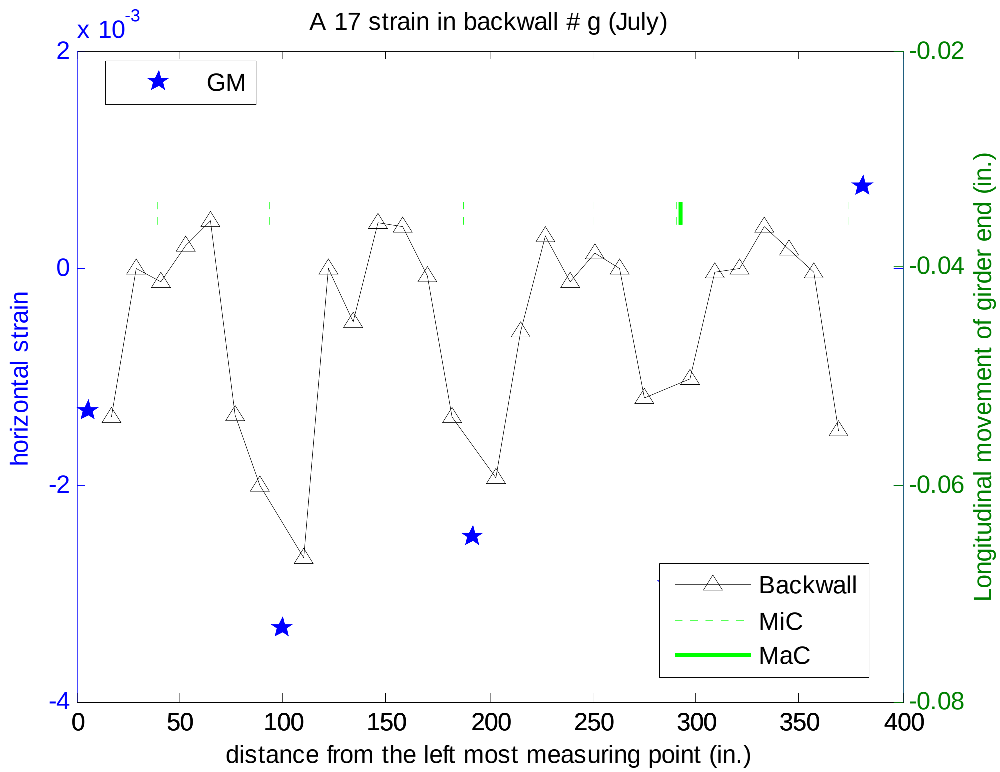


Figure C-51 Distribution of horizontal strain in backwall of Bridge A 1.7 in July 2007

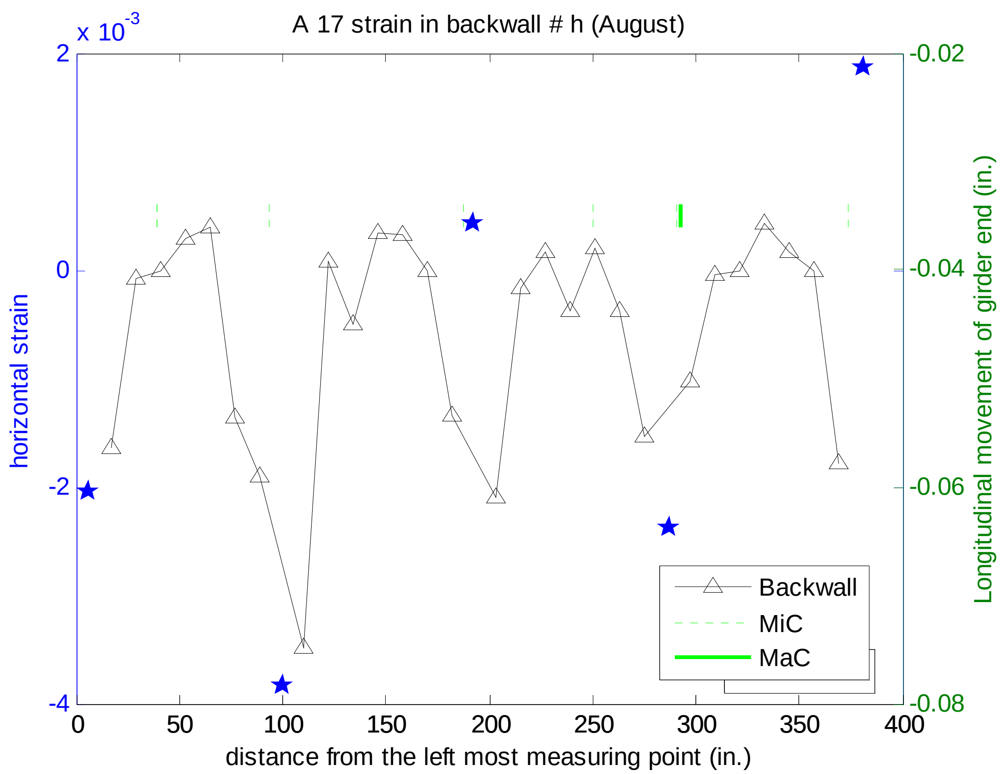


Figure C-52 Distribution of horizontal strain in backwall of Bridge A 1.7 in August 2007

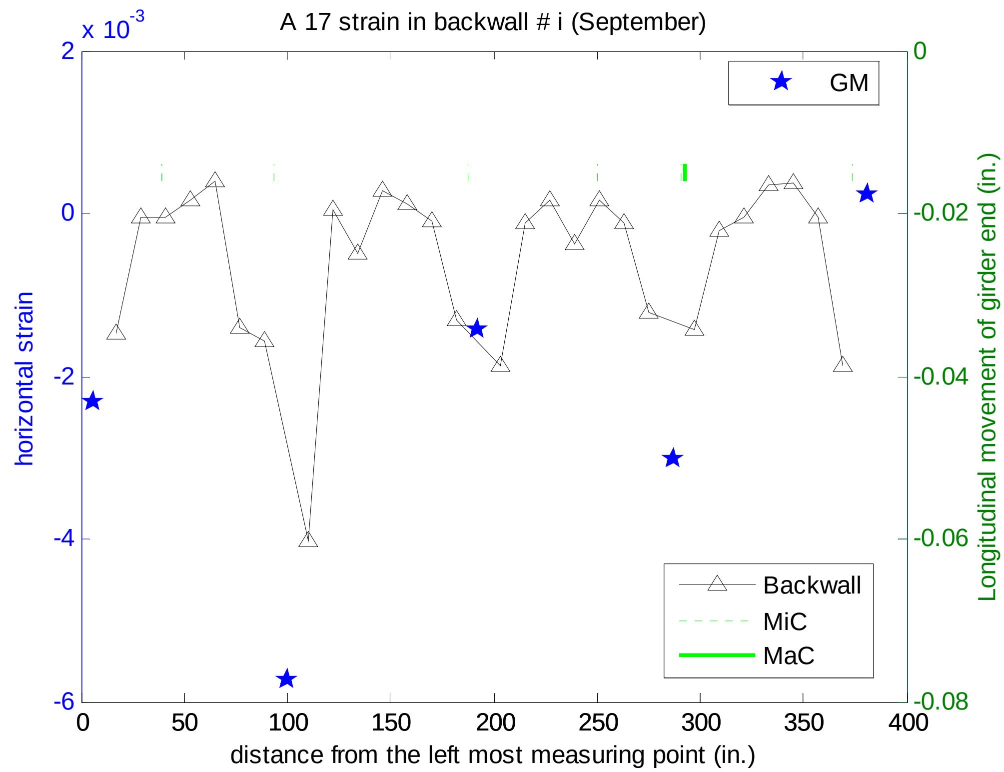


Figure C-53 Distribution of horizontal strain in backwall of Bridge A 1.7 in September 2007

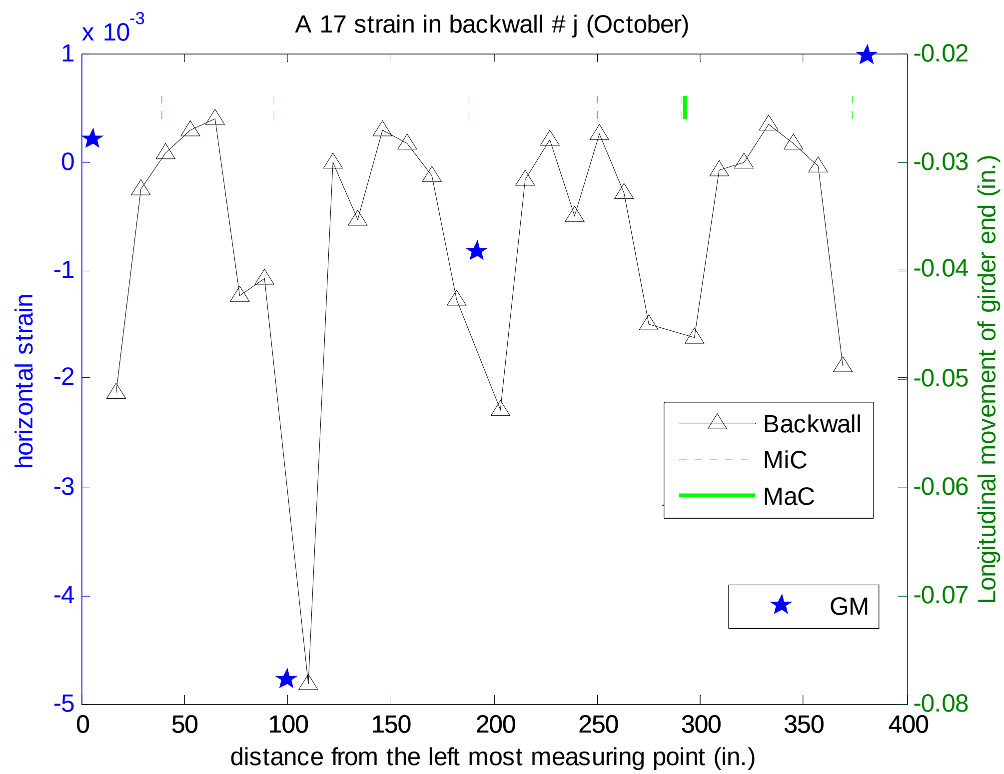


Figure C-54 Distribution of horizontal strain in backwall of Bridge A 1.7 in October 2007

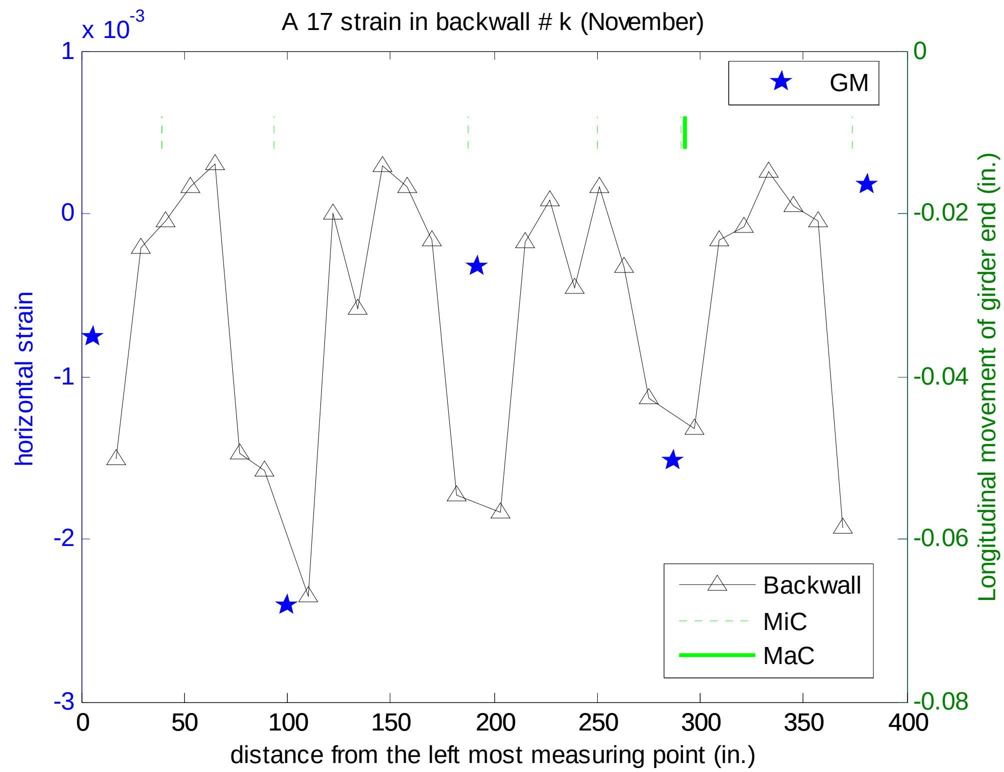


Figure C-55 Distribution of horizontal strain in backwall of Bridge A 1.7 in November 2007

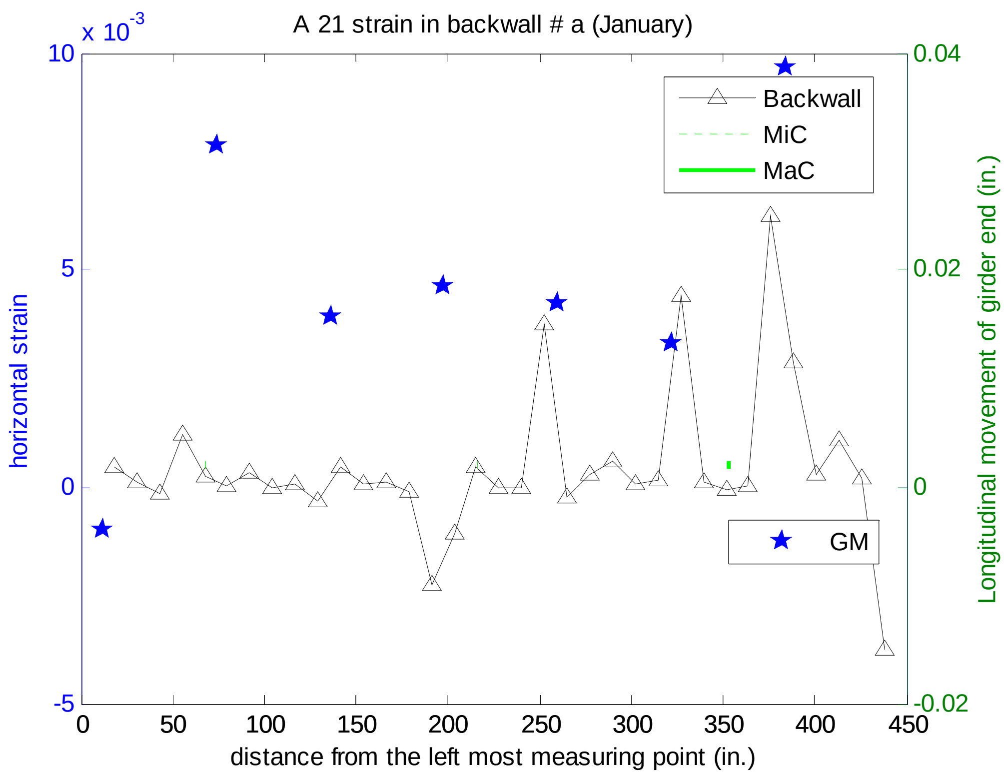


Figure C-56 Distribution of horizontal strain in backwall of Bridge A 2.1 in January 2007

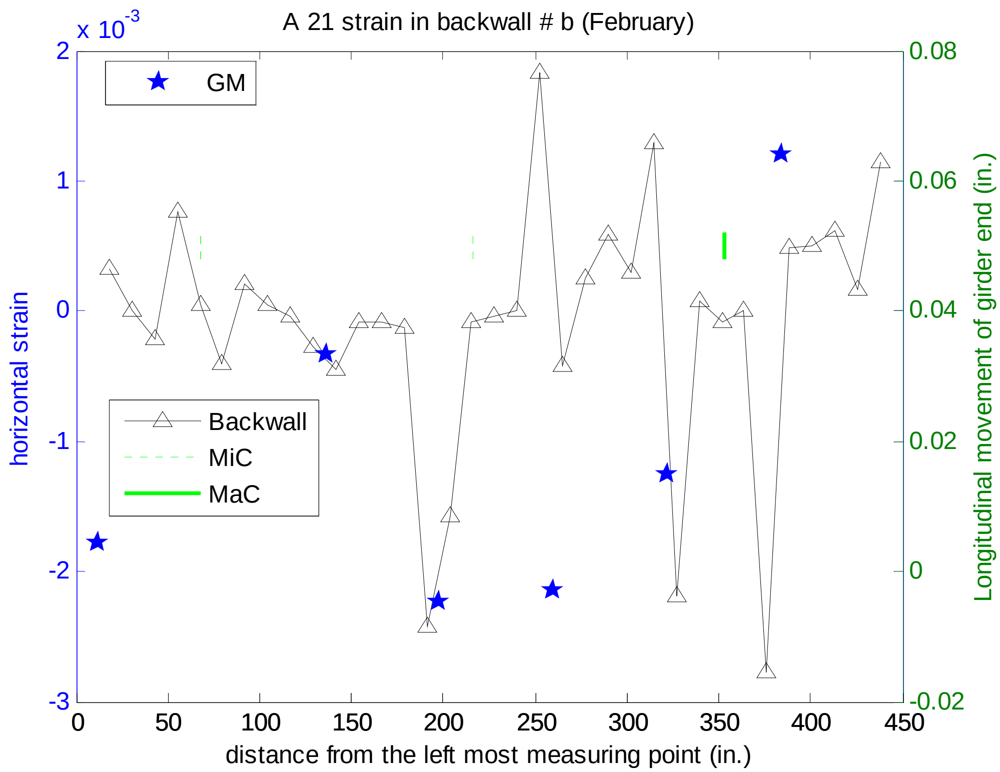


Figure C-57 Distribution of horizontal strain in backwall of Bridge A 2.1 in February 2007

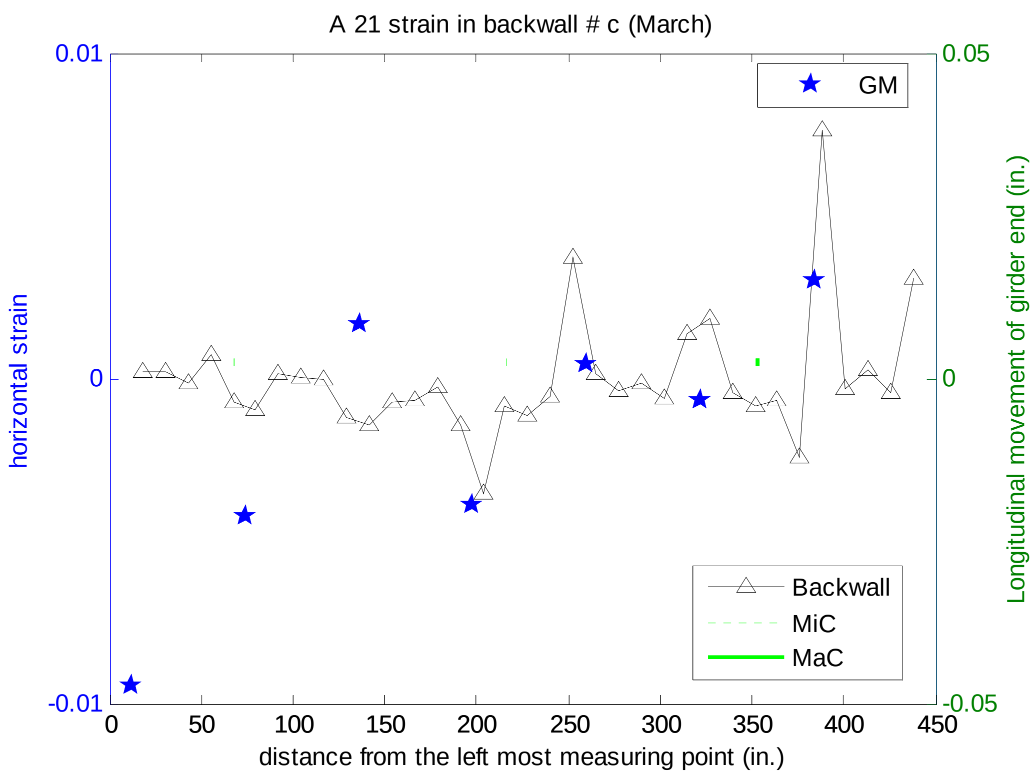


Figure C-58 Distribution of horizontal strain in backwall of Bridge A 2.1 in March 2007

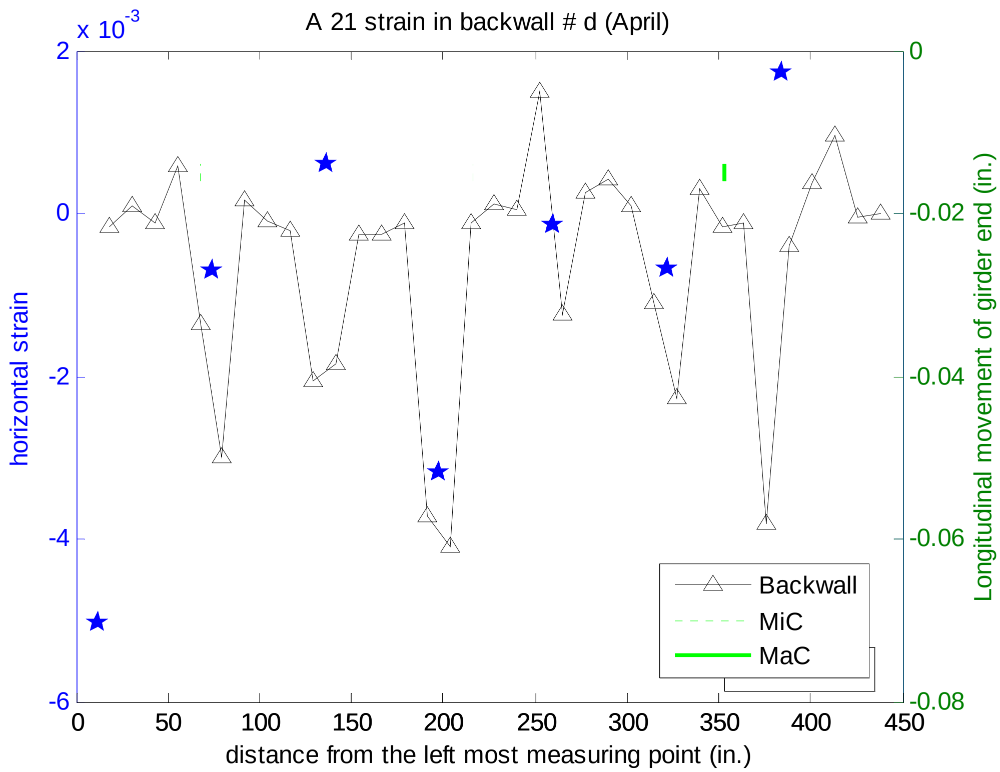


Figure C-59 Distribution of horizontal strain in backwall of Bridge A 2.1 in April 2007

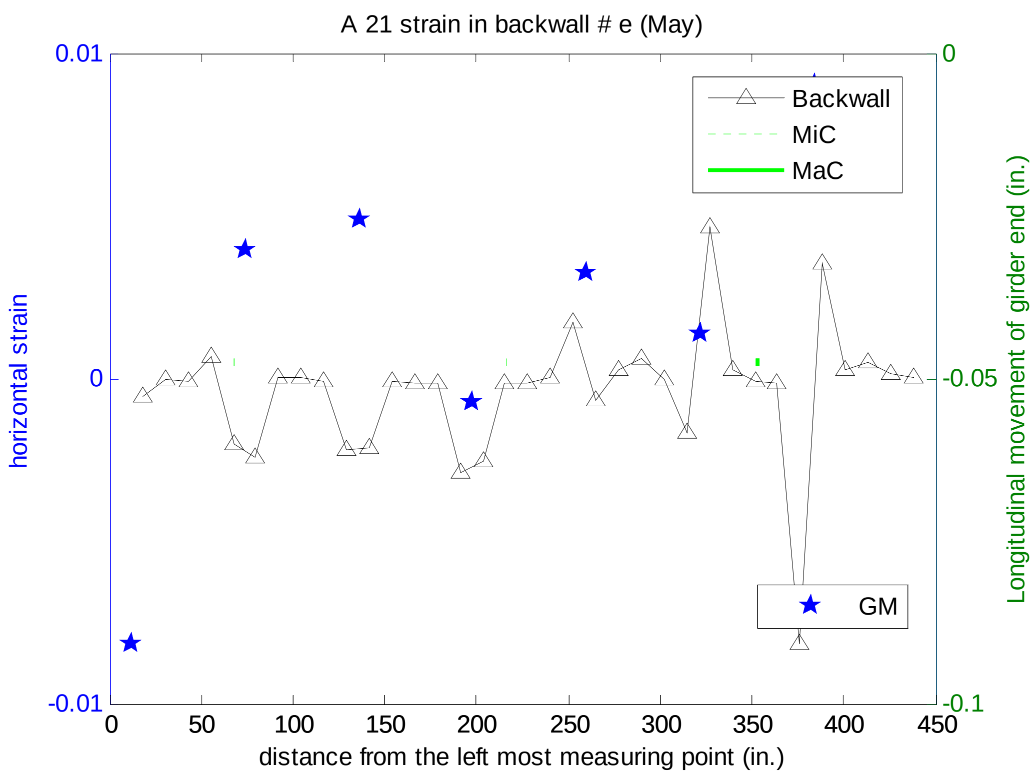


Figure C-60 Distribution of horizontal strain in backwall of Bridge A 2.1 in May 2007

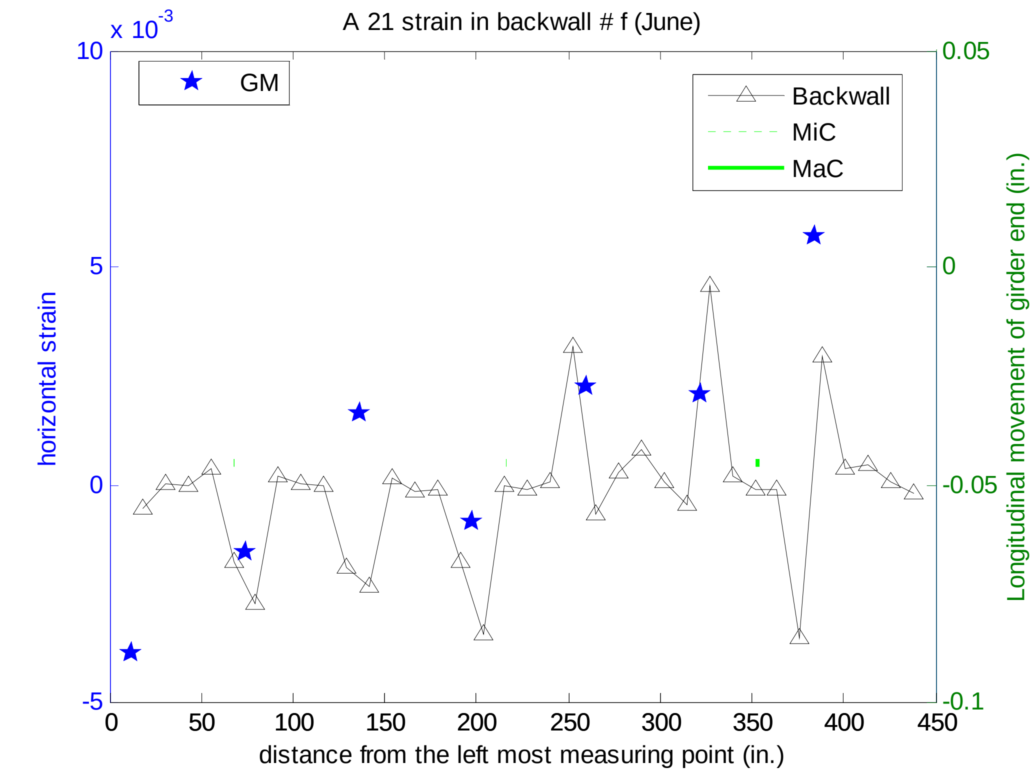


Figure C-61 Distribution of horizontal strain in backwall of Bridge A 2.1 in June 2007

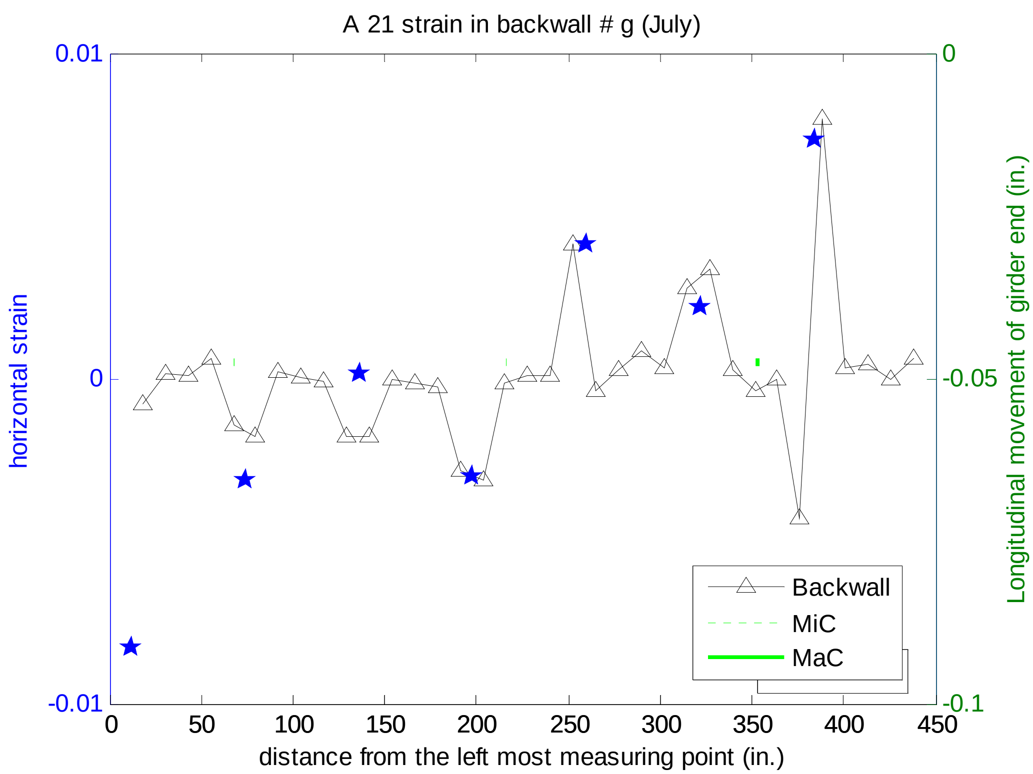


Figure C-62 Distribution of horizontal strain in backwall of Bridge A 2.1 in July 2007

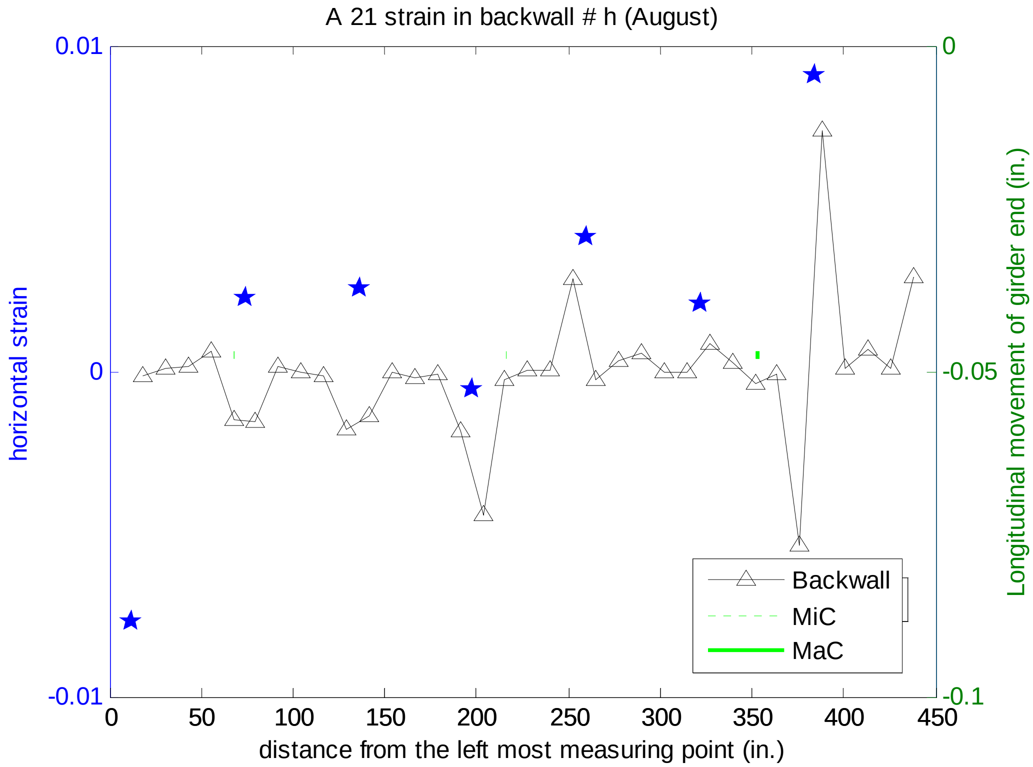


Figure C-63 Distribution of horizontal strain in backwall of Bridge A 2.1 in August 2007

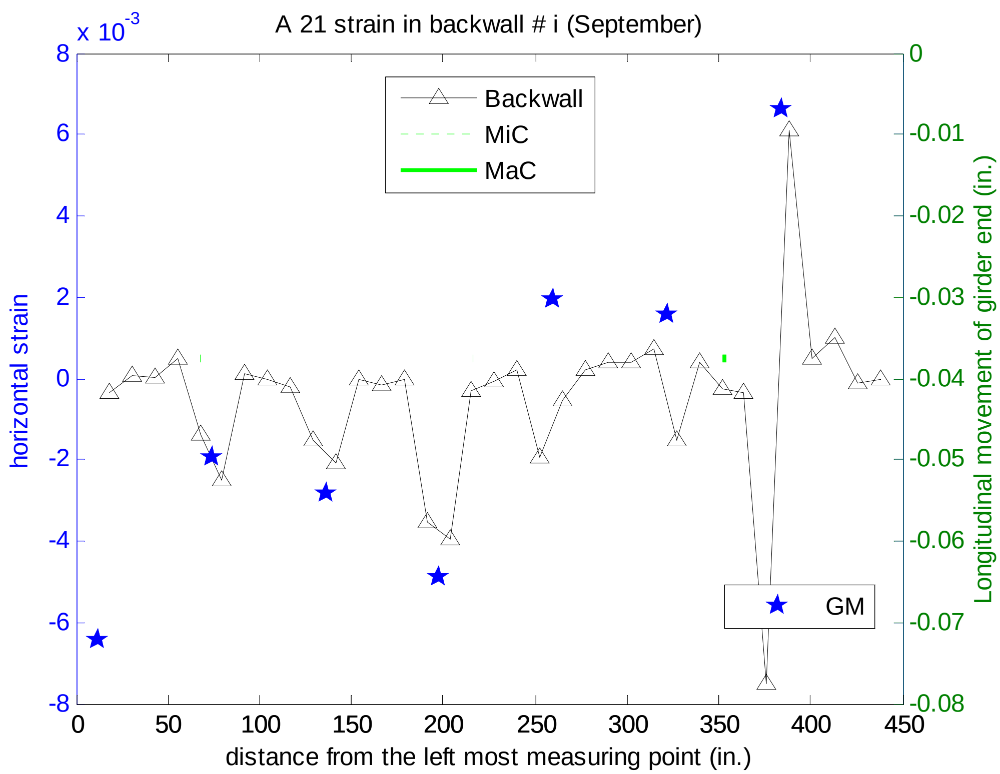


Figure C-64 Distribution of horizontal strain in backwall of Bridge A 2.1 in September 2007

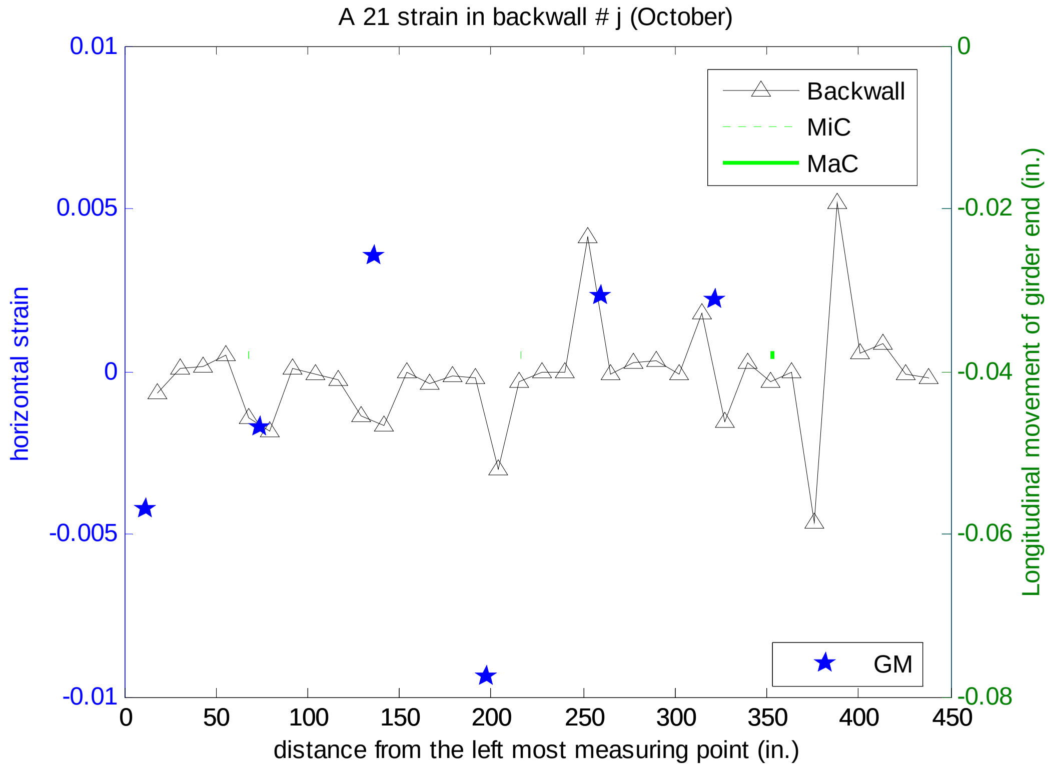


Figure C-65 Distribution of horizontal strain in backwall of Bridge A 2.1 in October 2007

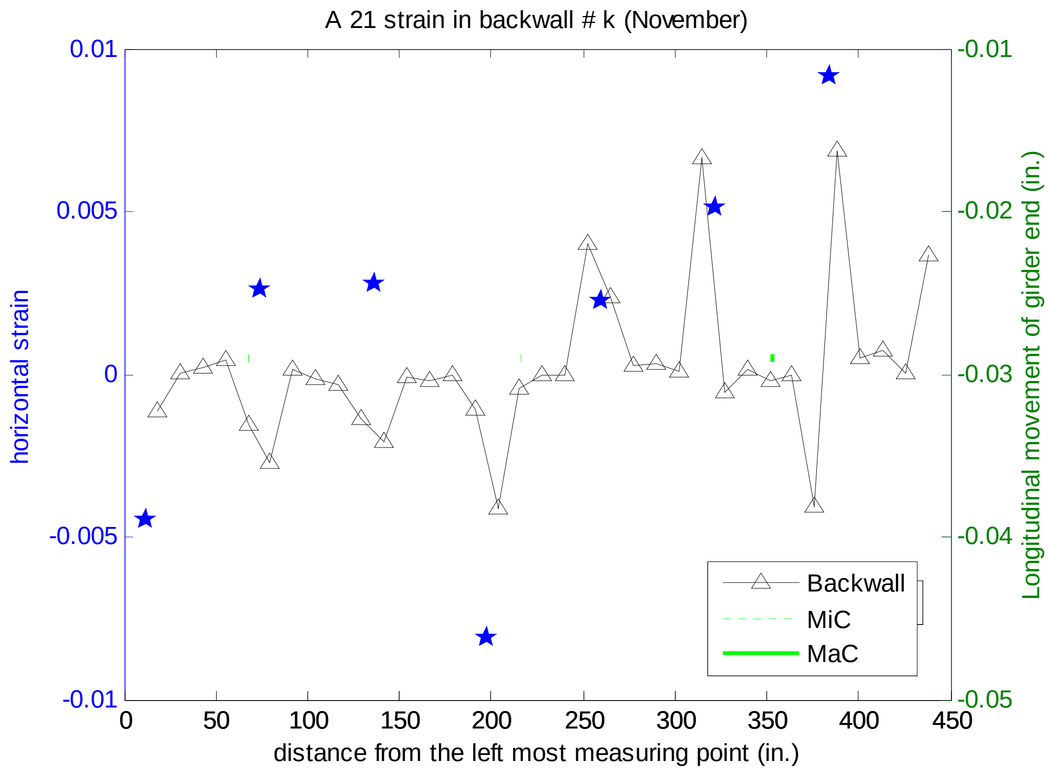


Figure C-66 Distribution of horizontal strain in backwall of Bridge A 2.1 in November 2007

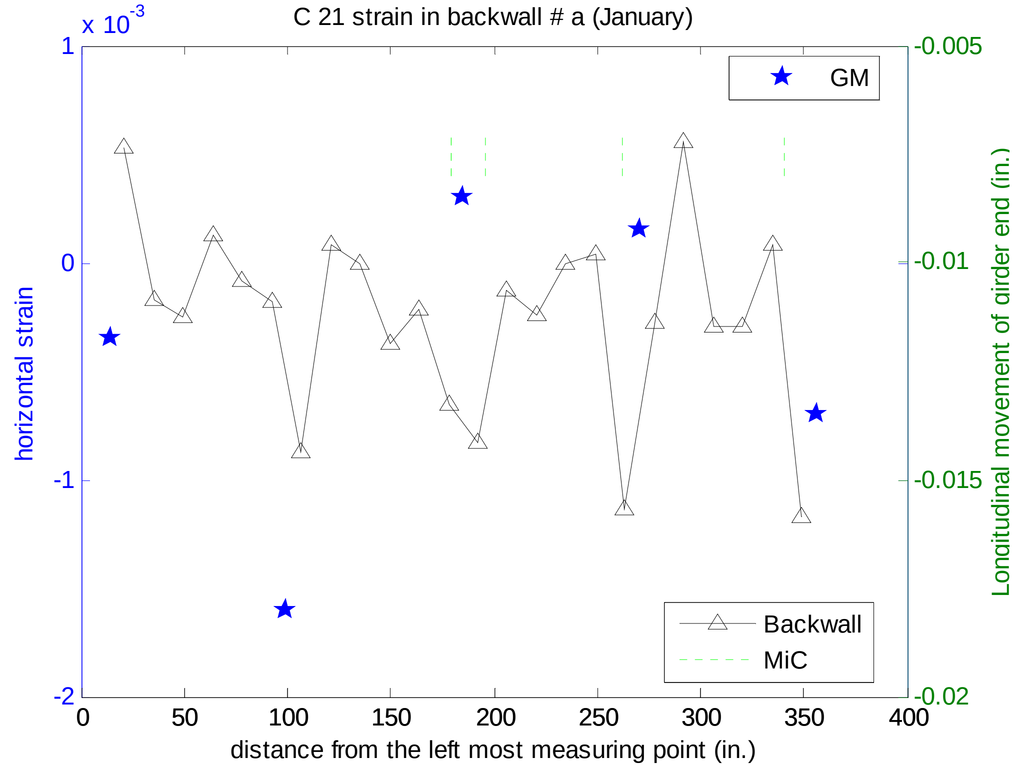


Figure C-67 Distribution of horizontal strain in backwall of Bridge C 2.1 in January 2007

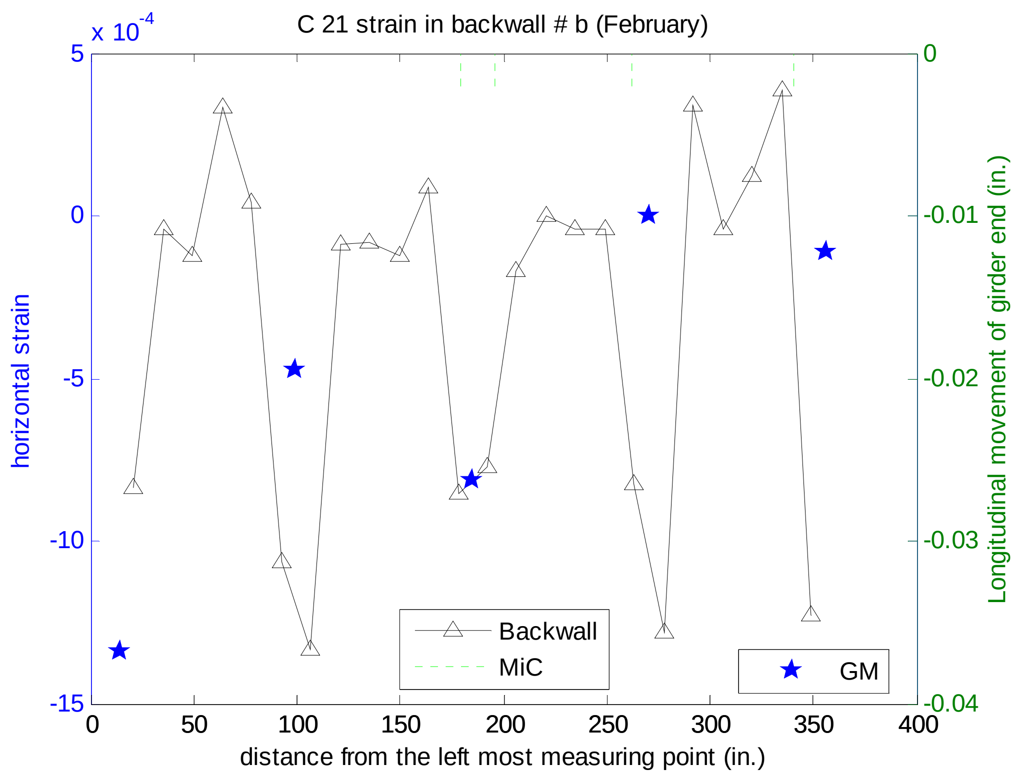


Figure C-68 Distribution of horizontal strain in backwall of Bridge C 2.1 in February 2007

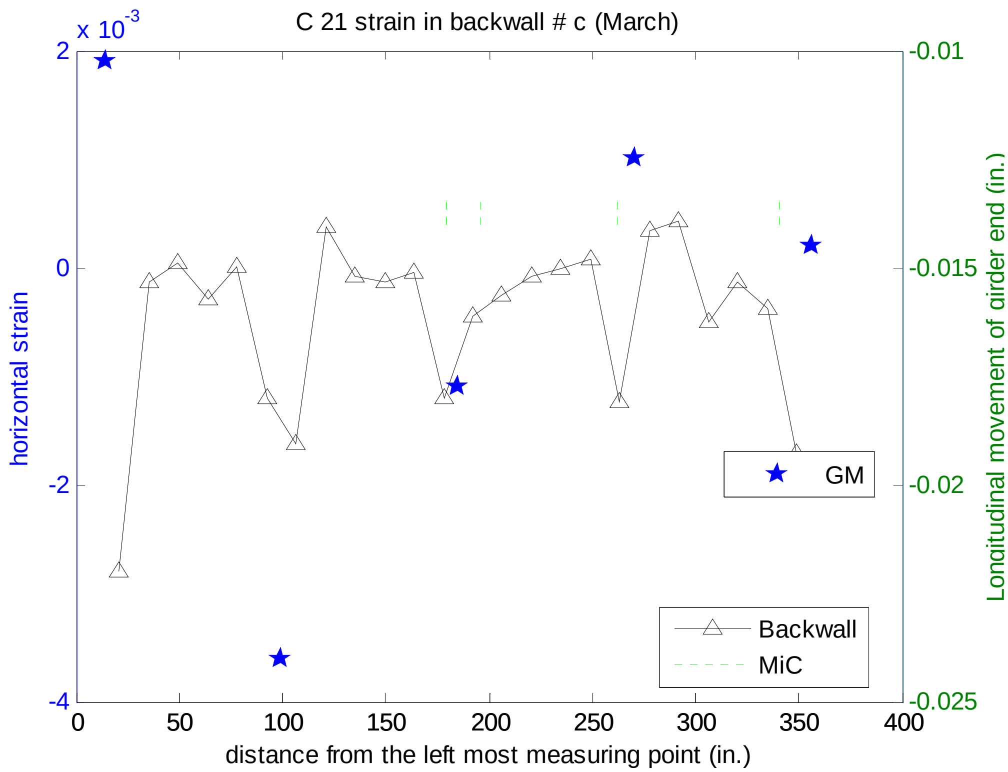


Figure C-69 Distribution of horizontal strain in backwall of Bridge C 2.1 in March 2007

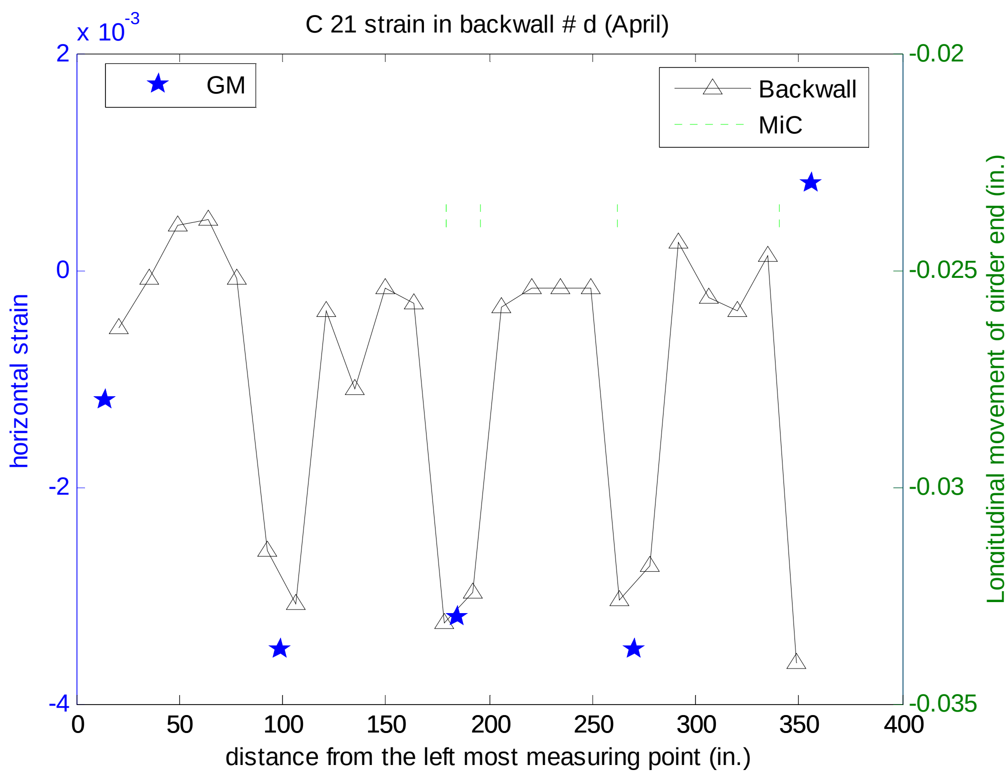


Figure C-70 Distribution of horizontal strain in backwall of Bridge C 2.1 in April 2007

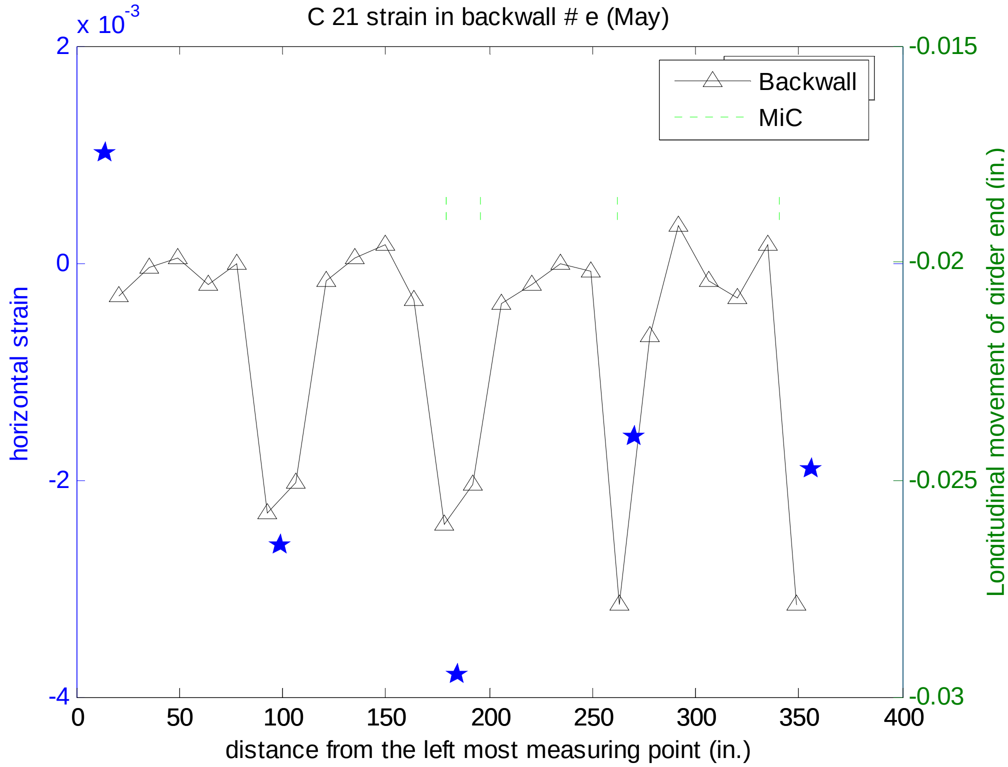


Figure C-71 Distribution of horizontal strain in backwall of Bridge C 2.1 in May 2007

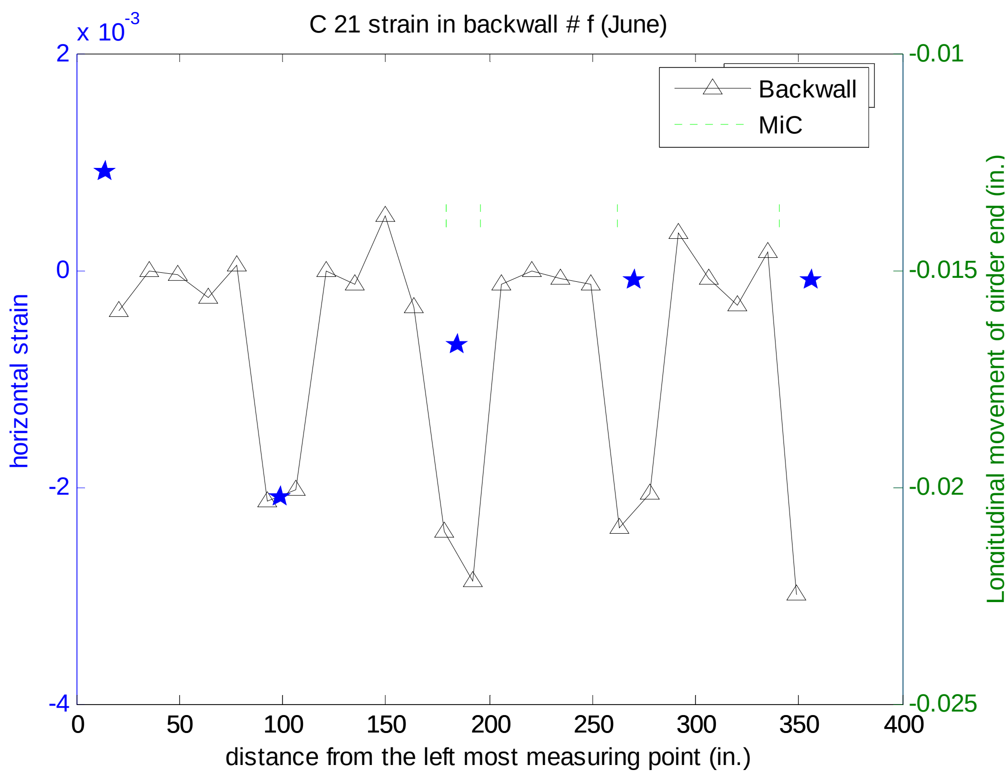


Figure C-72 Distribution of horizontal strain in backwall of Bridge C 2.1 in June 2007

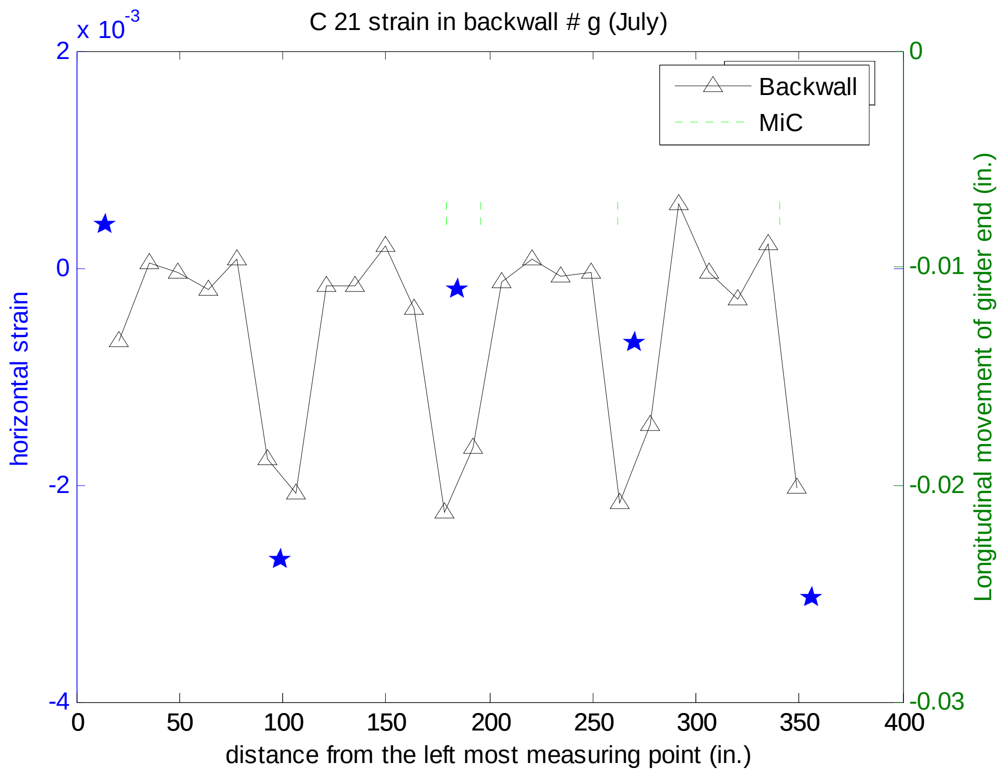


Figure C-73 Distribution of horizontal strain in backwall of Bridge C 2.1 in July 2007

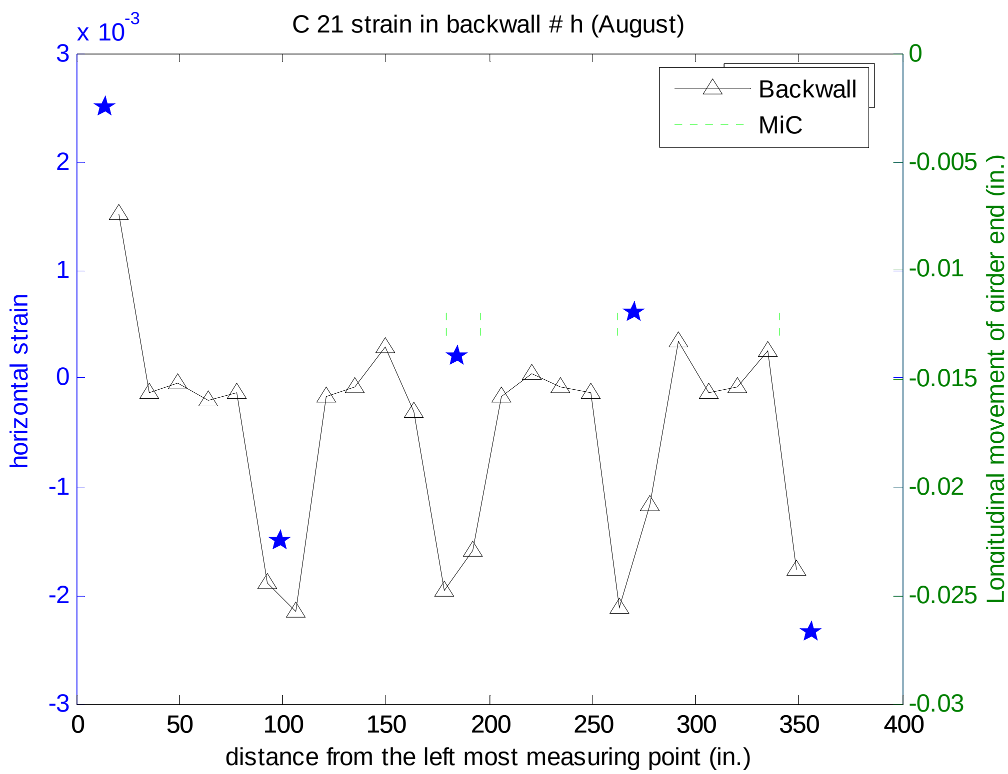


Figure C-74 Distribution of horizontal strain in backwall of Bridge C 2.1 in August 2007

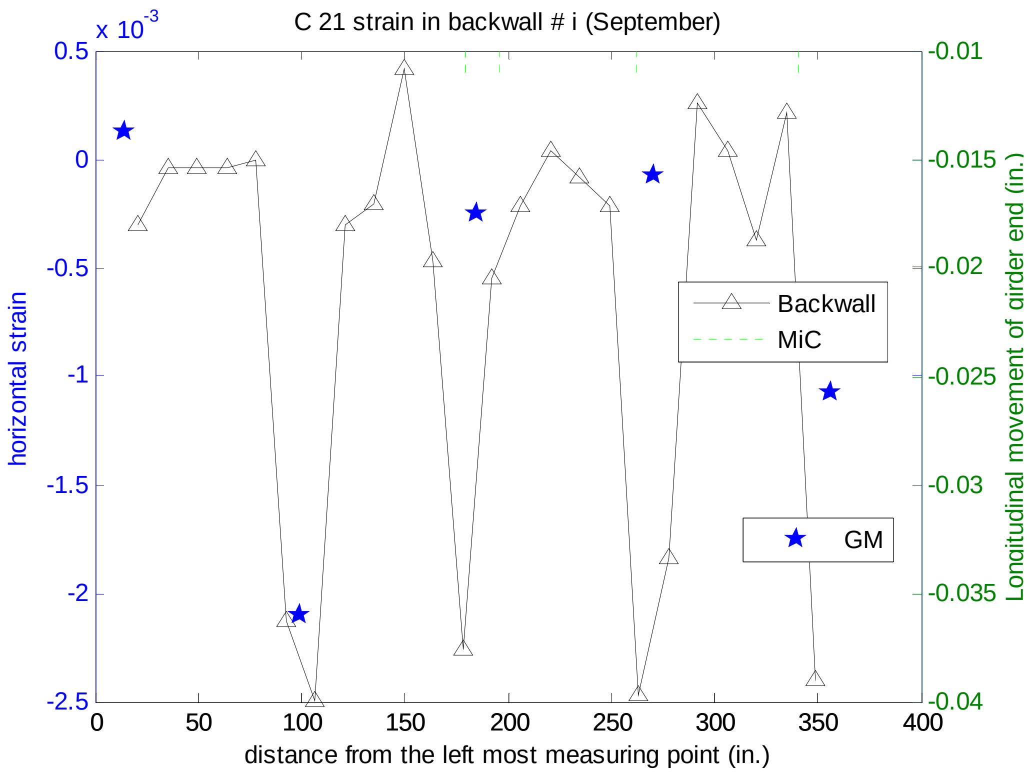


Figure C-75 Distribution of horizontal strain in backwall of Bridge C 2.1 in September 2007

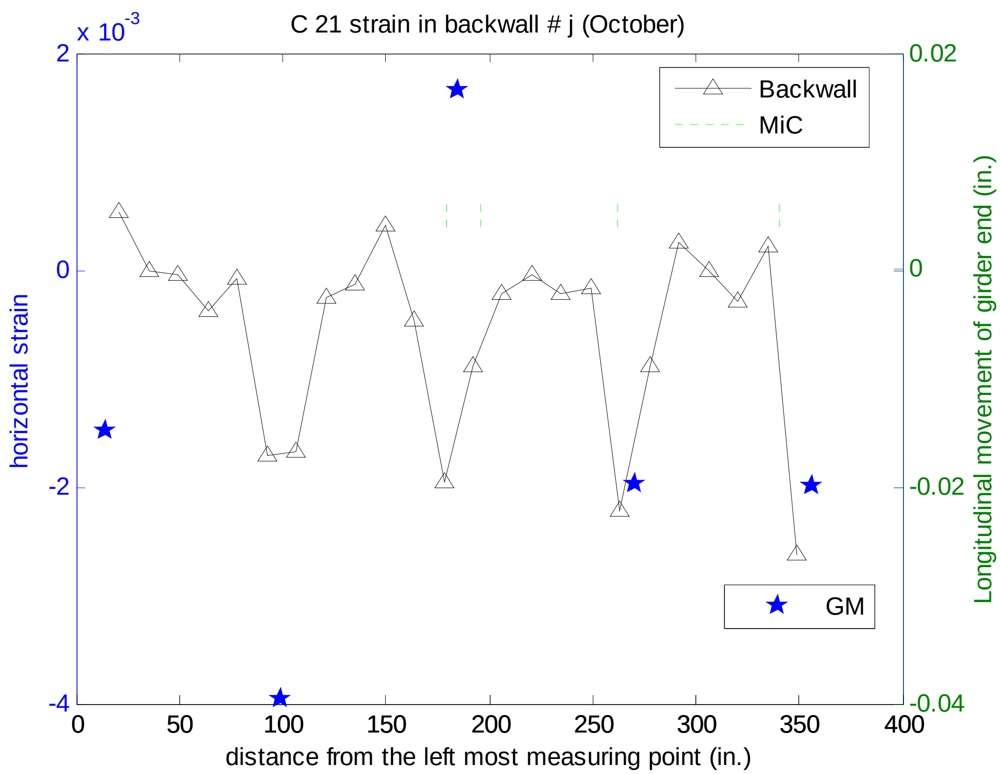


Figure C-76 Distribution of horizontal strain in backwall of Bridge C 2.1 in October 2007

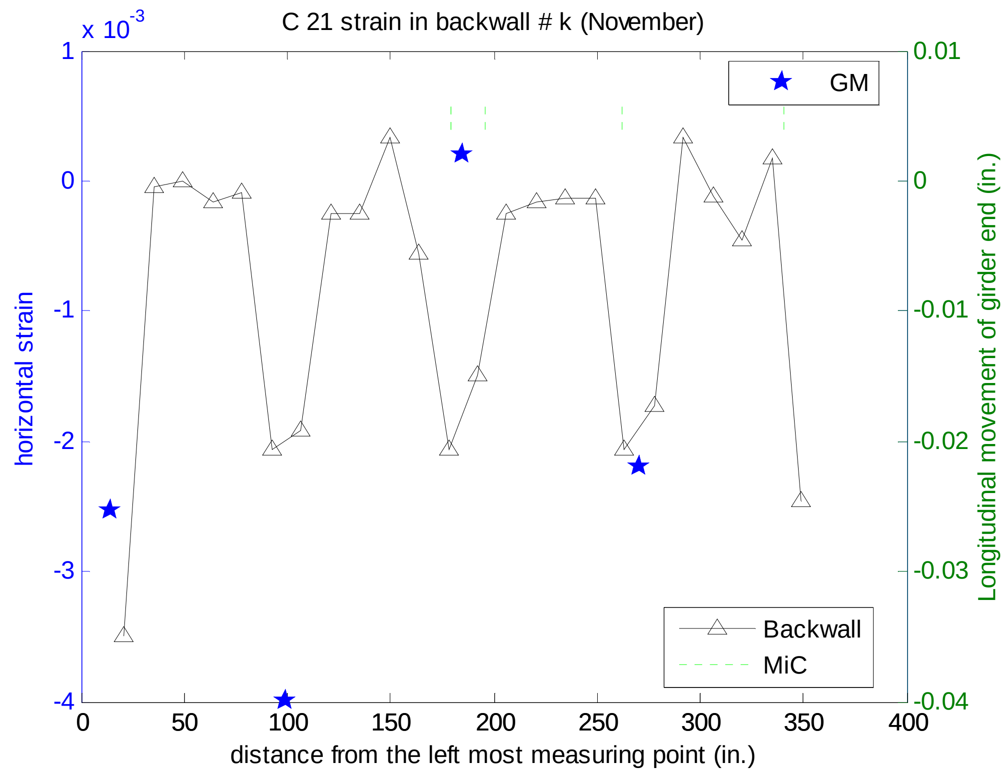


Figure C-77 Distribution of horizontal strain in backwall of Bridge C 2.1 in November 2007

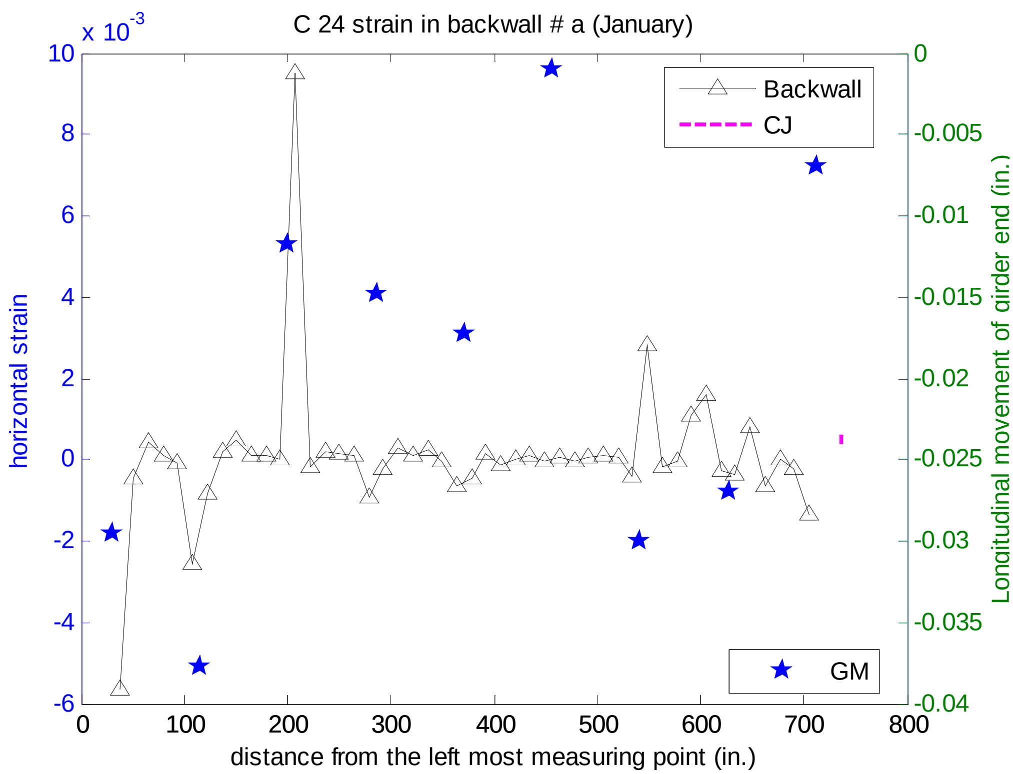


Figure C-78 Distribution of horizontal strain in backwall of Bridge C 2.4 in January 2007

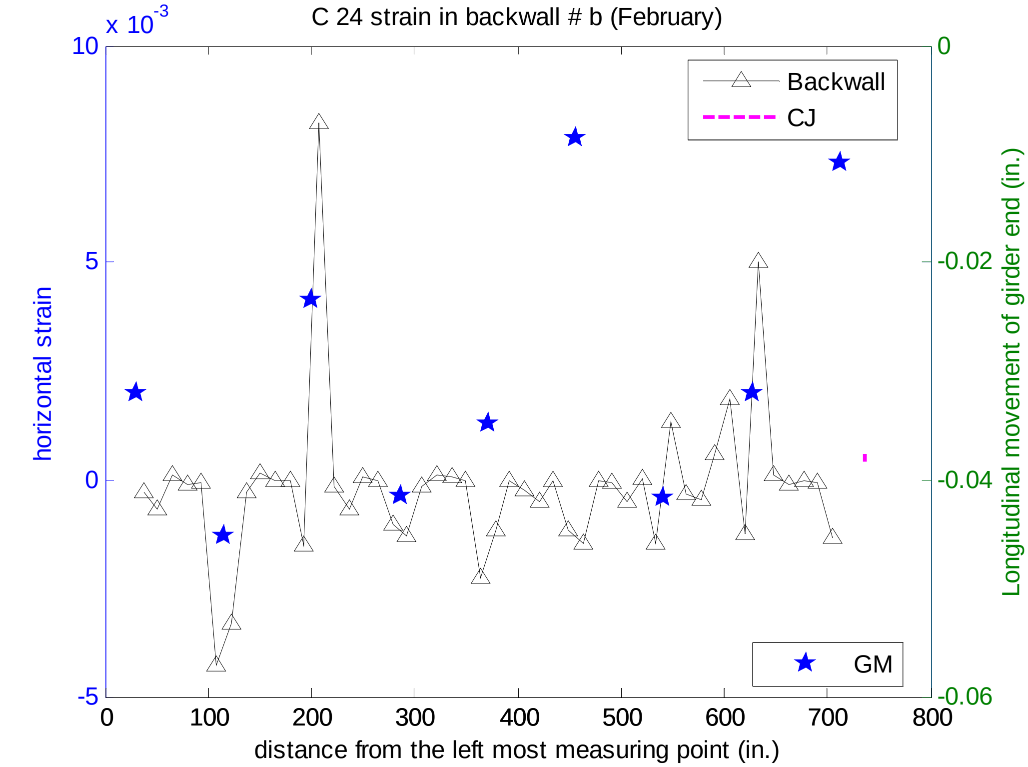


Figure C-79 Distribution of horizontal strain in backwall of Bridge C 2.4 in February 2007

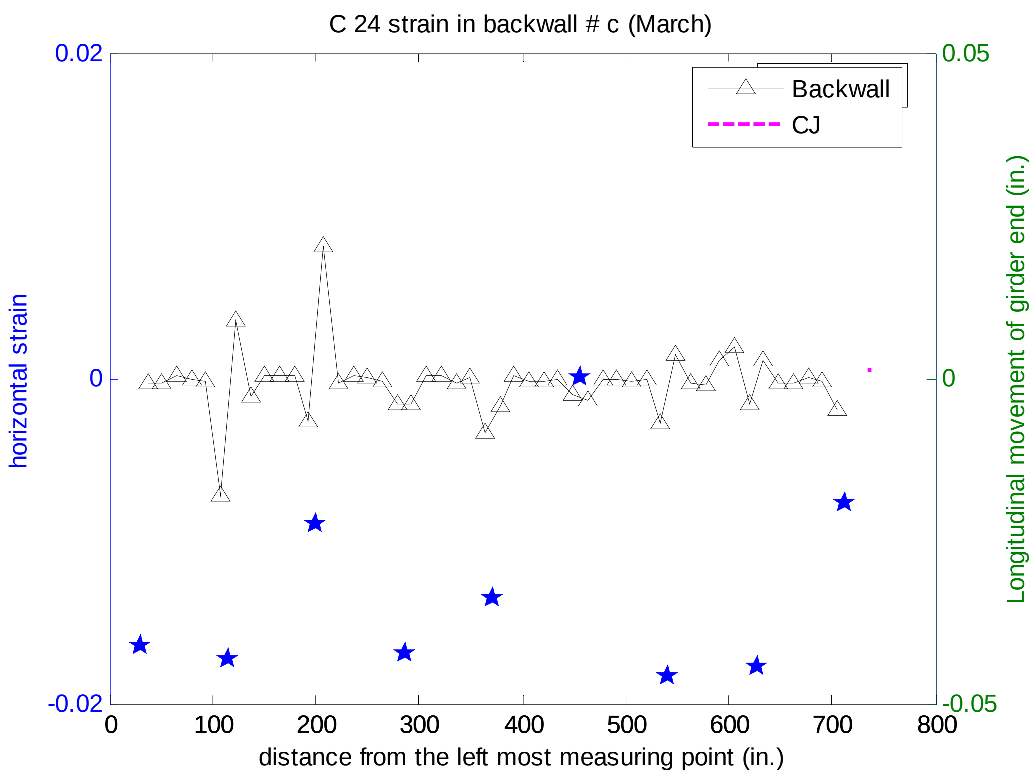


Figure C-80 Distribution of horizontal strain in backwall of Bridge C 2.4 in March 2007

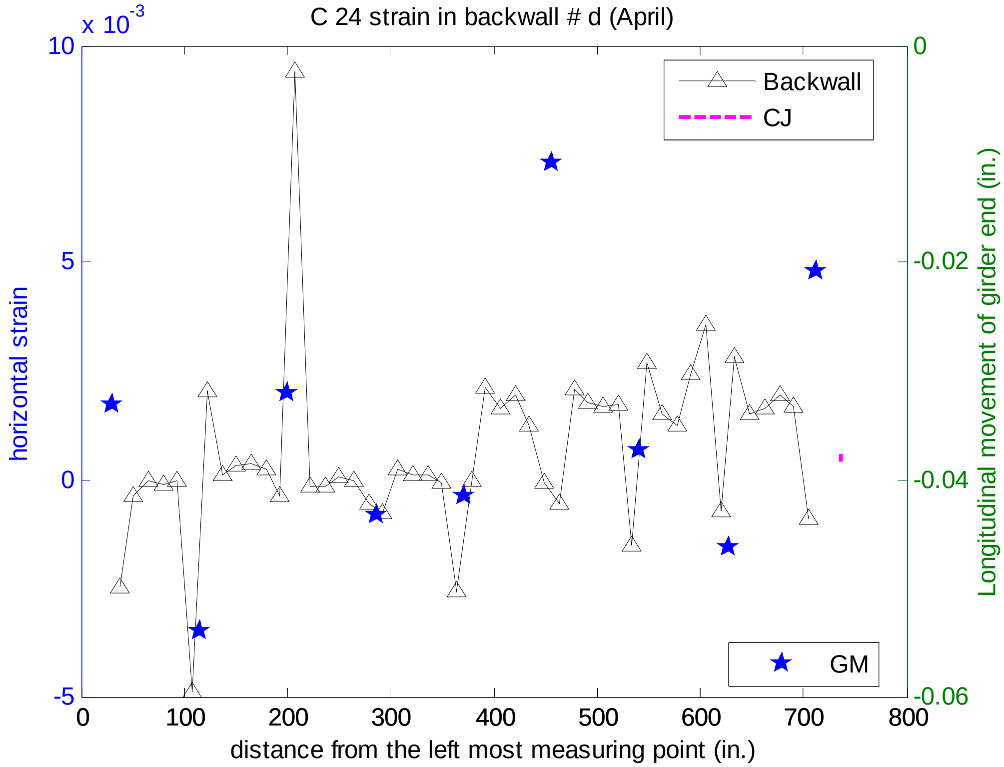


Figure C-81 Distribution of horizontal strain in backwall of Bridge C 2.4 in April 2007

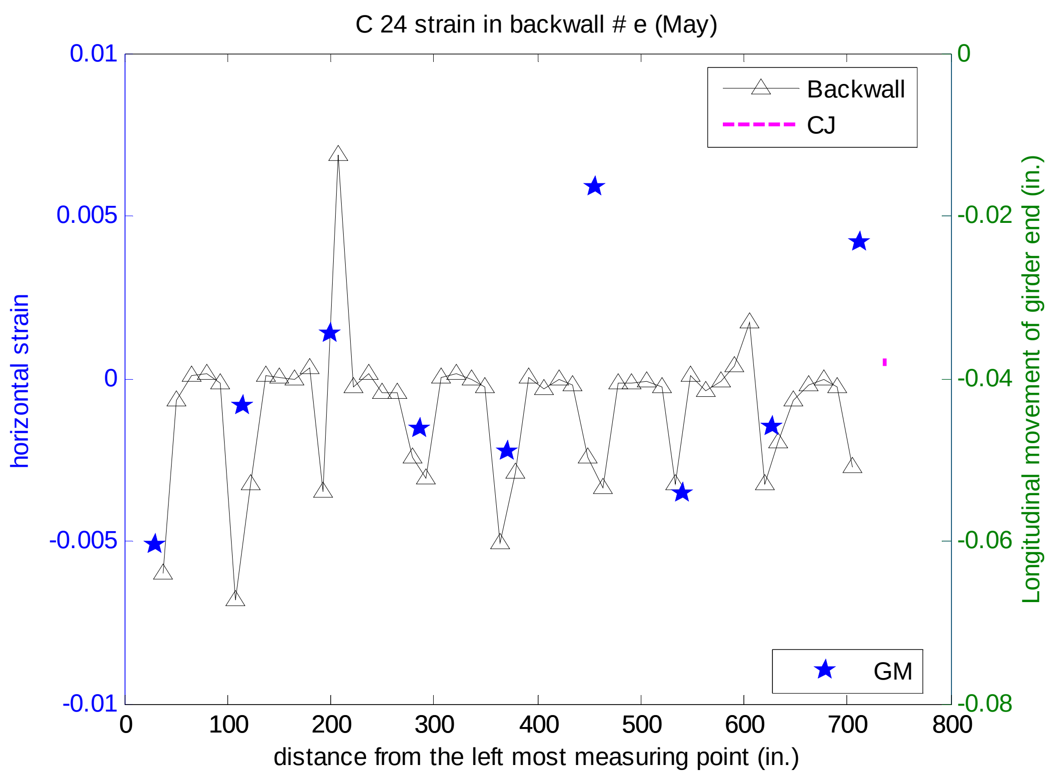


Figure C-82 Distribution of horizontal strain in backwall of Bridge C 2.4 in May 2007

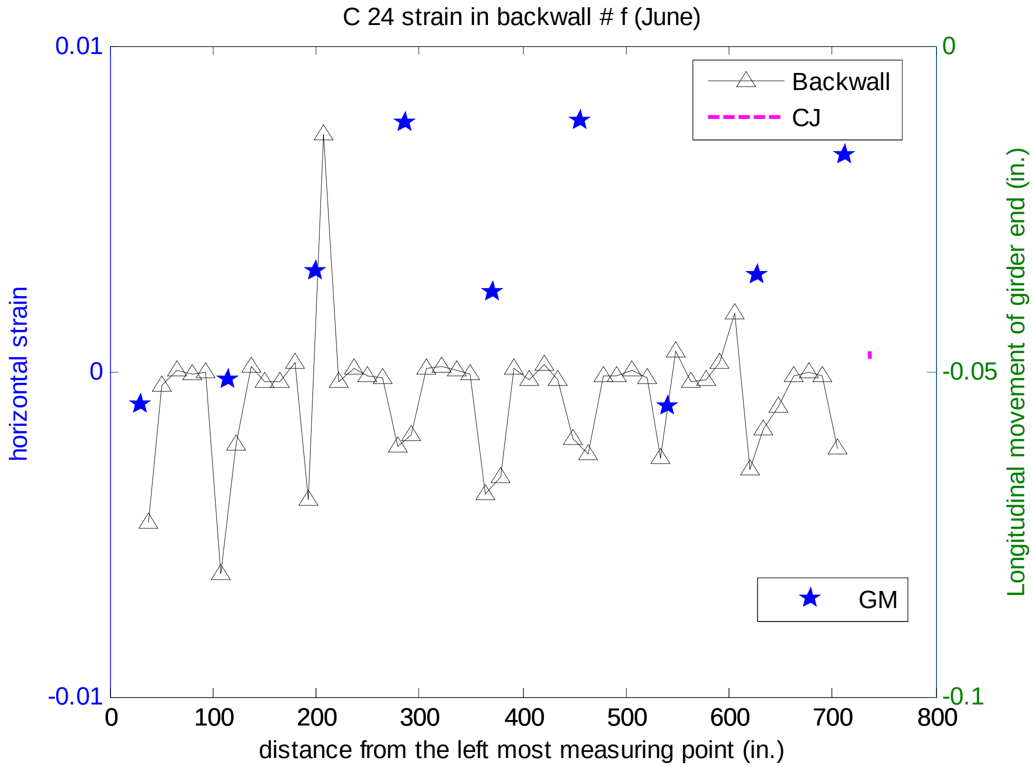


Figure C-83 Distribution of horizontal strain in backwall of Bridge C 2.4 in June 2007

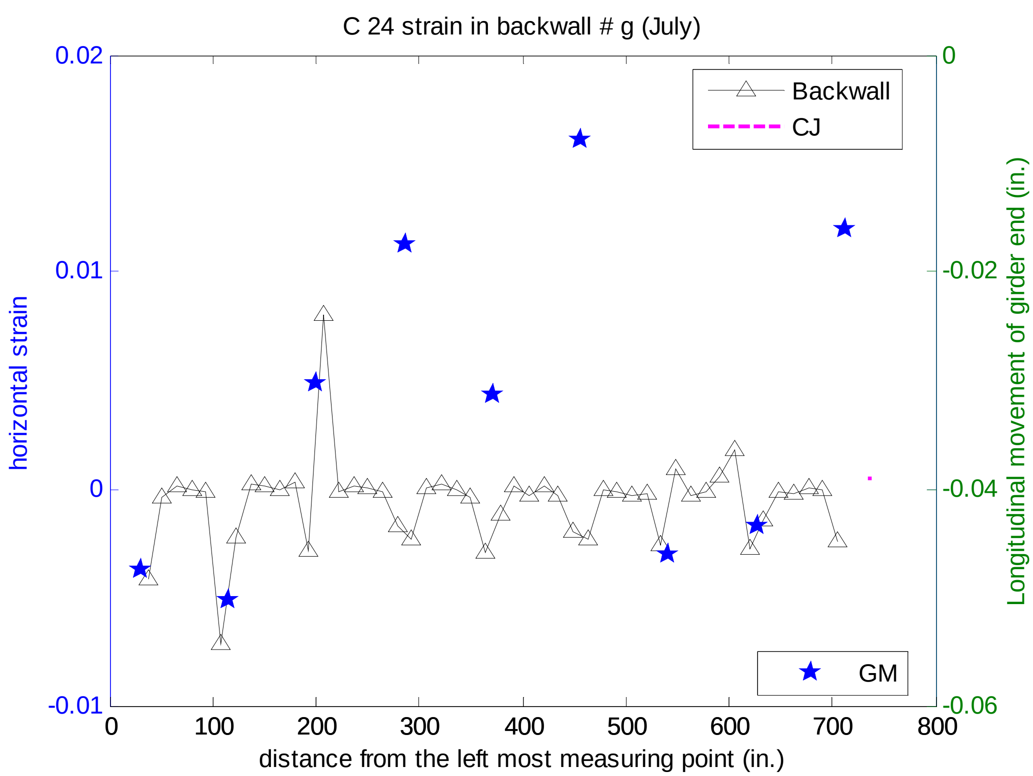


Figure C-84 Distribution of horizontal strain in backwall of Bridge C 2.4 in July 2007

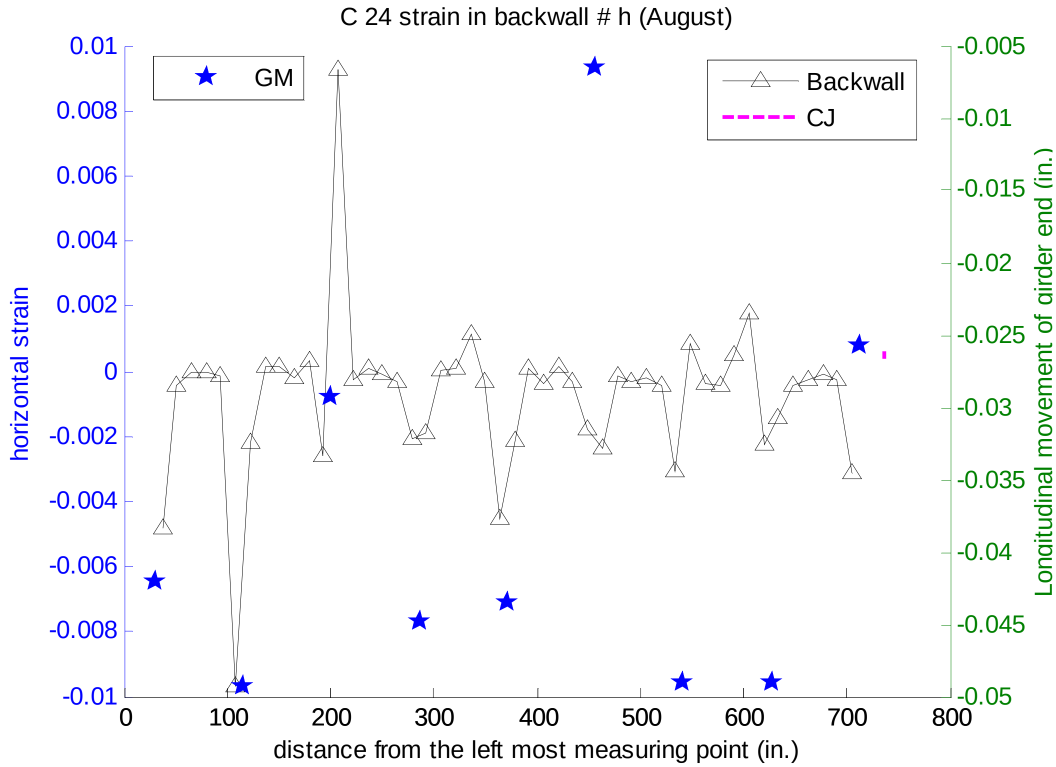


Figure C-85 Distribution of horizontal strain in backwall of Bridge C 2.4 in August 2007

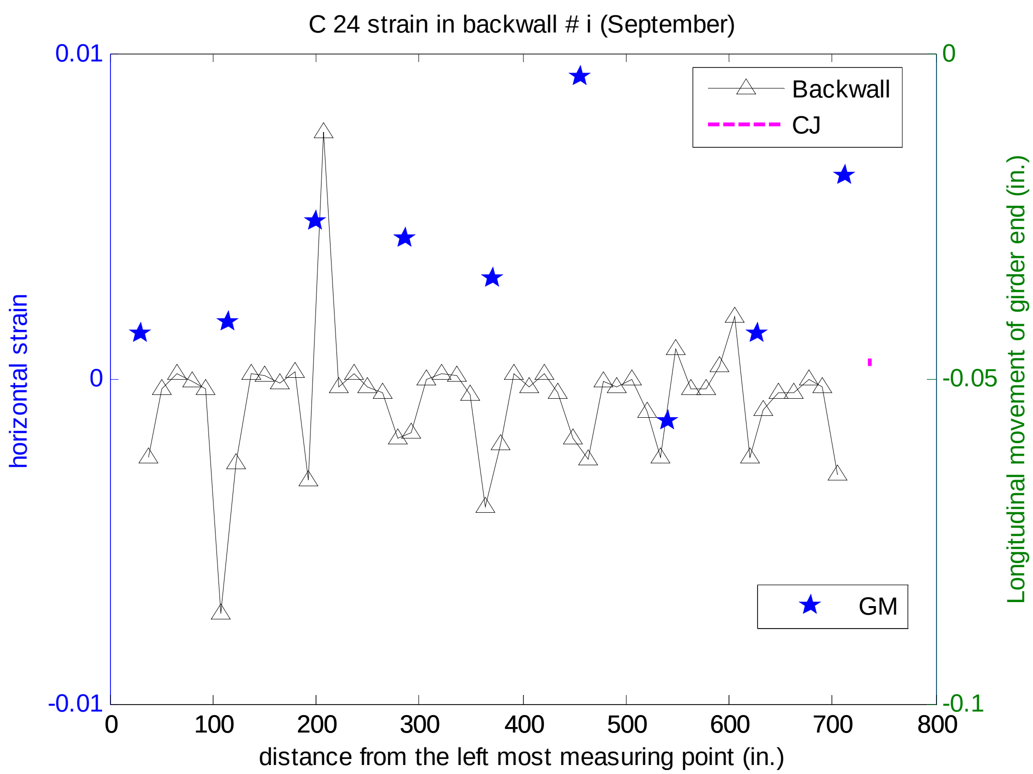


Figure C-86 Distribution of horizontal strain in backwall of Bridge C 2.4 in September 2007

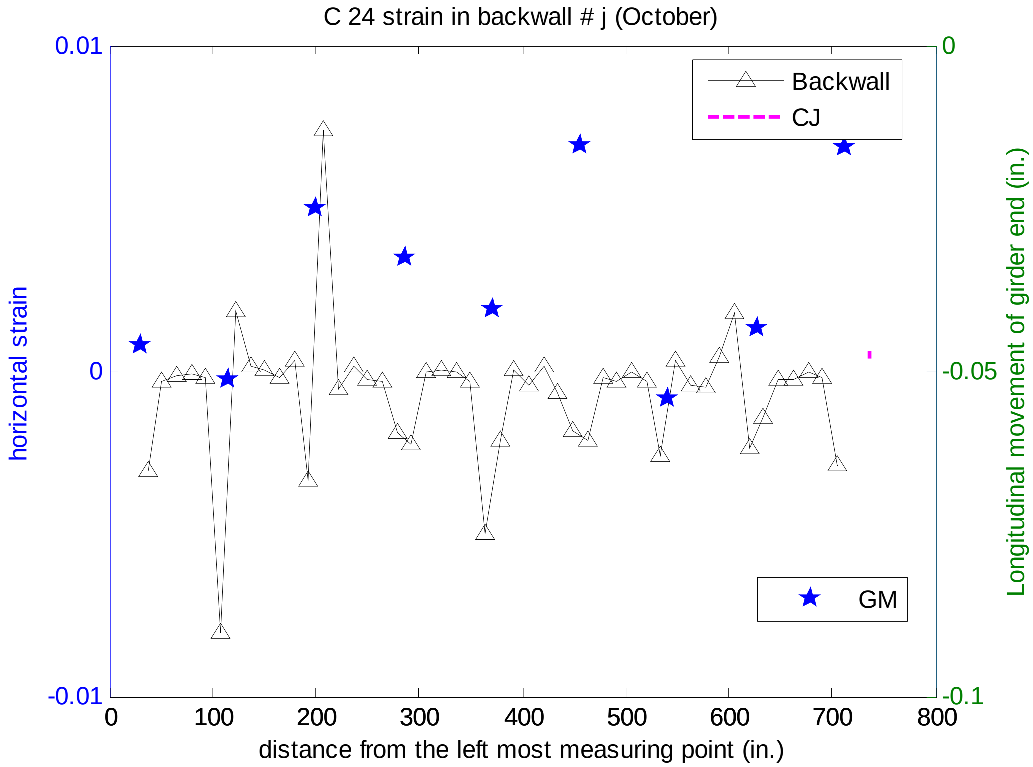


Figure C-87 Distribution of horizontal strain in backwall of Bridge C 2.4 in October 2007

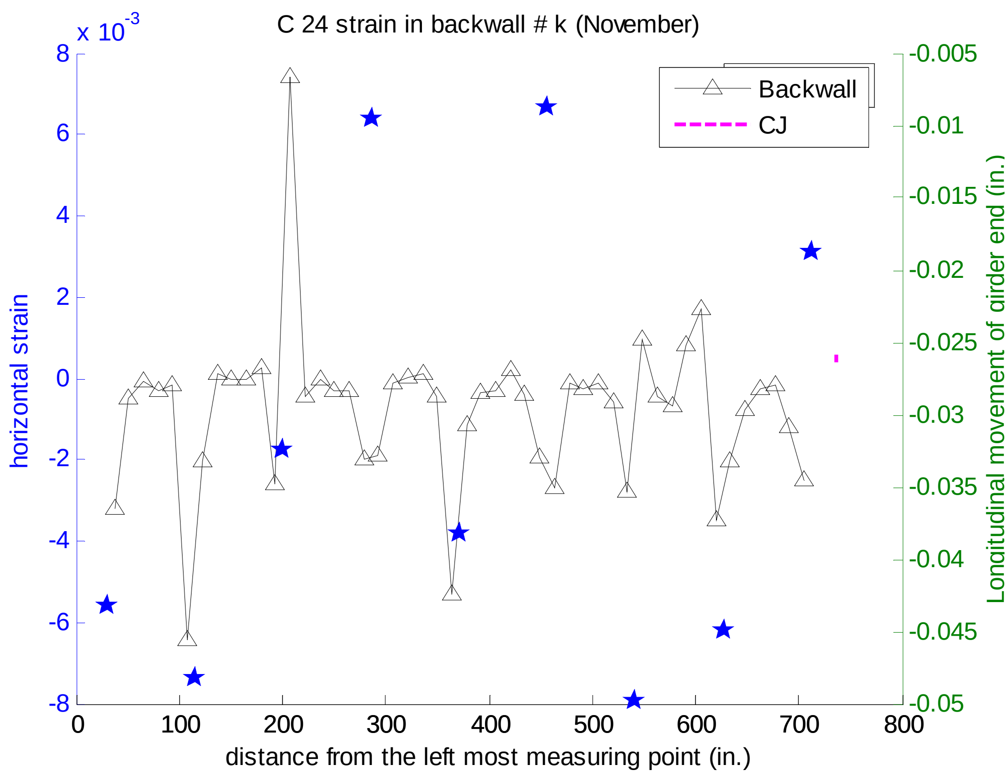


Figure C-88 Distribution of horizontal strain in backwall of Bridge C 2.4 in November 2007

C.II Peak Strain vs. Time and Temperature in Region around Girders

Variation of maximum and minimum strain in the regions in abutment wall of A 1.7 is plotted in Figure C-89 to **Figure** C-93. Similarly, variation of maximum and minimum strain in the regions in abutment wall of A 2.1, C 2.1, and C 2.4 is plotted in **Figure** C-94 to **Figure** C-100, **Figure** C-101 to **Figure** C-105, and **Figure** C-106 to **Figure** C-114, respectively.

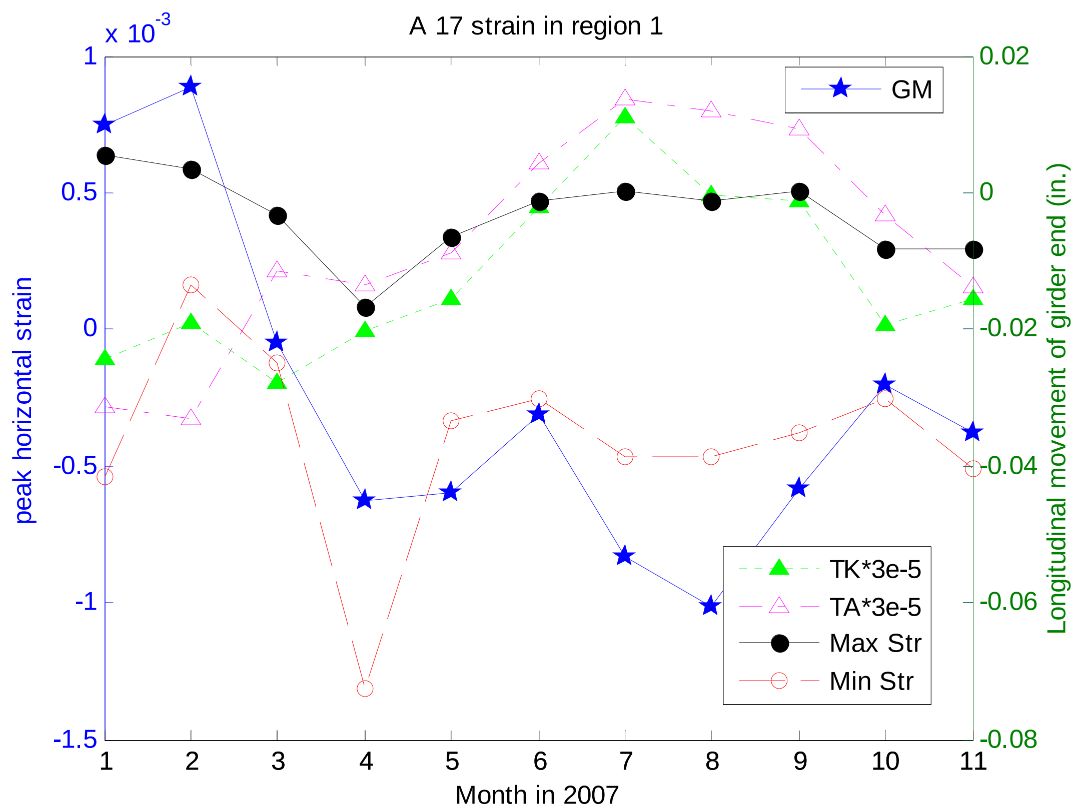


Figure C-89 Variation of peak strain in region 1 of Bridge A 1.7 with time and temperature

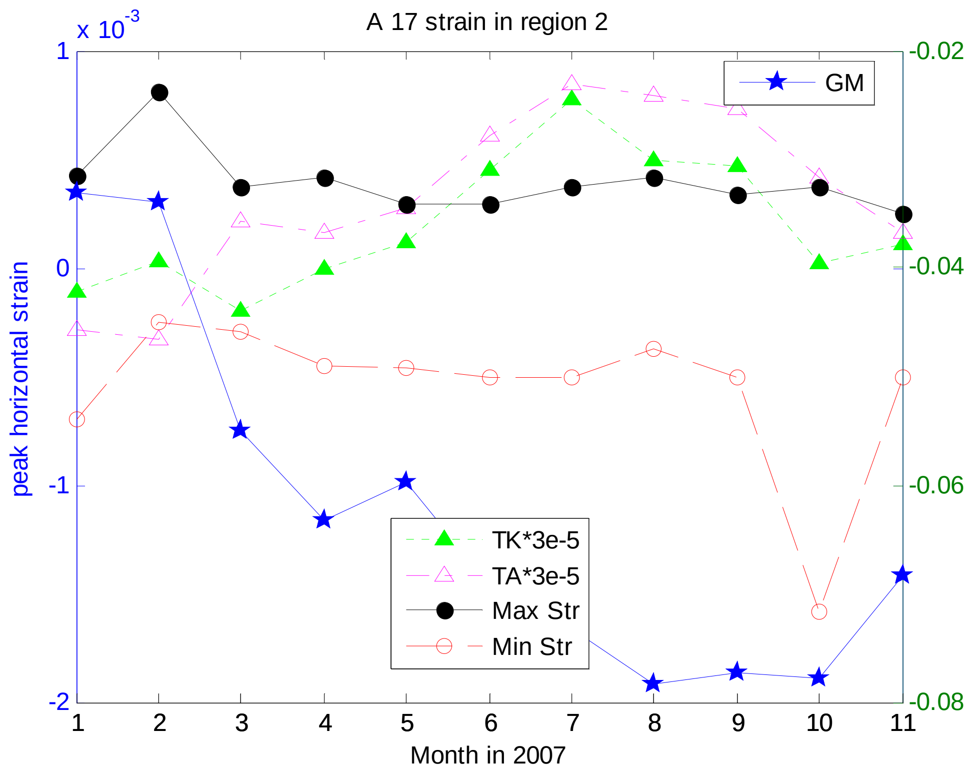


Figure C-90 Variation of peak strain in region 2 of Bridge A 1.7 with time and temperature

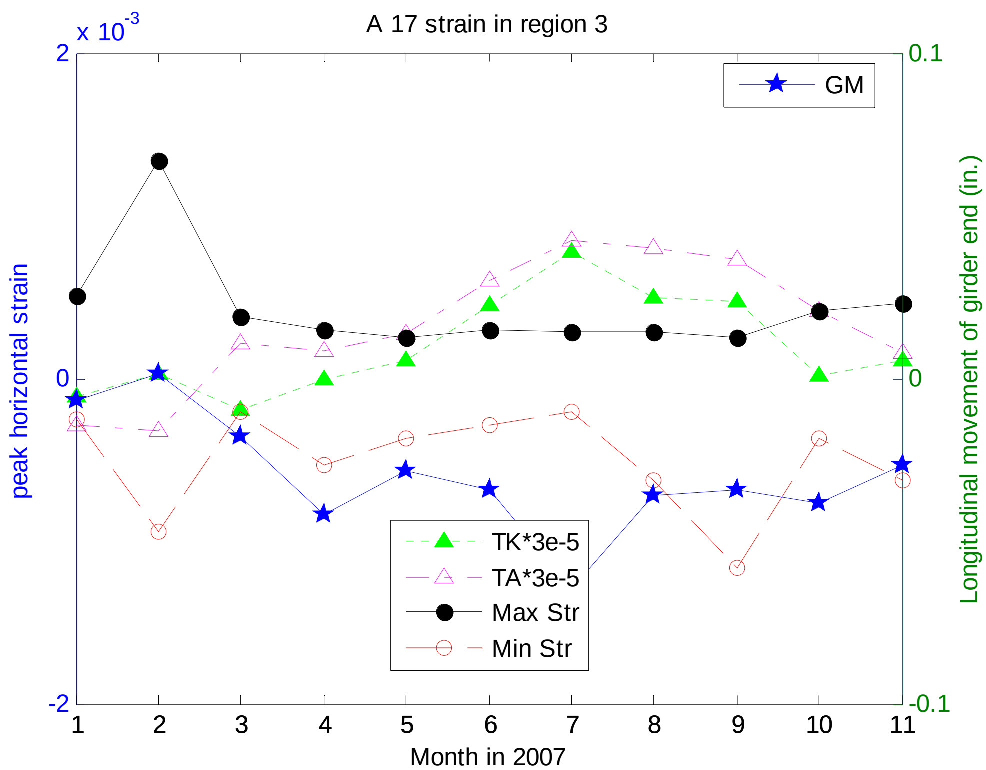


Figure C-91 Variation of peak strain in region 3 of Bridge A 1.7 with time and temperature

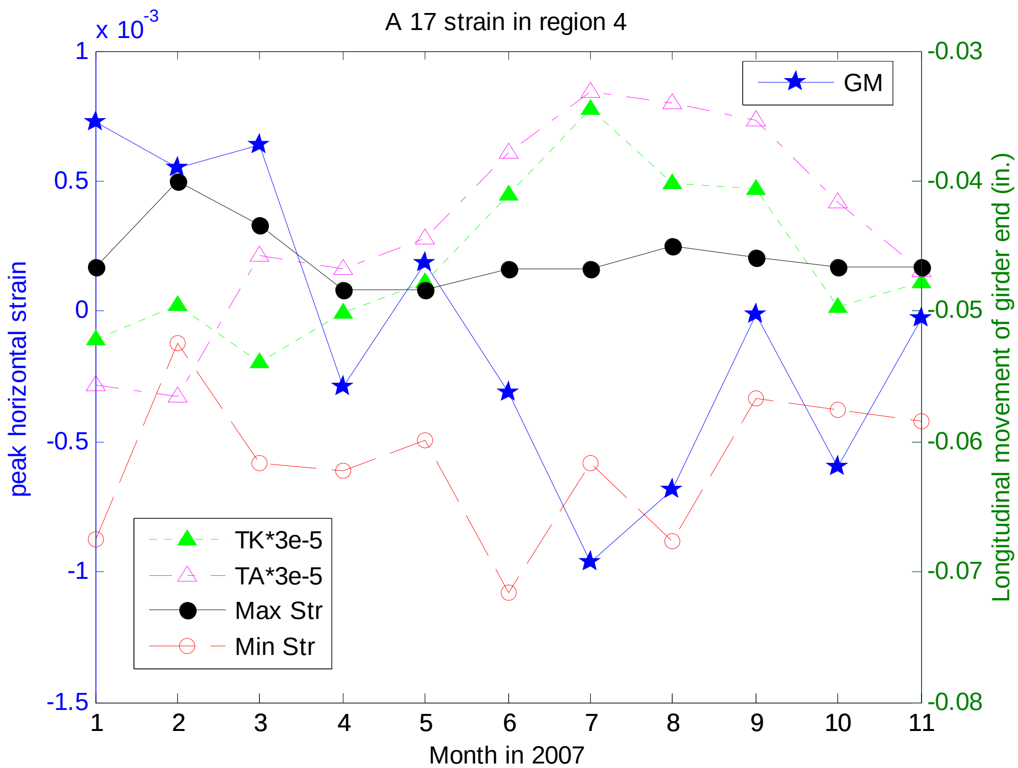


Figure C-92 Variation of peak strain in region 4 of Bridge A 1.7 with time and temperature

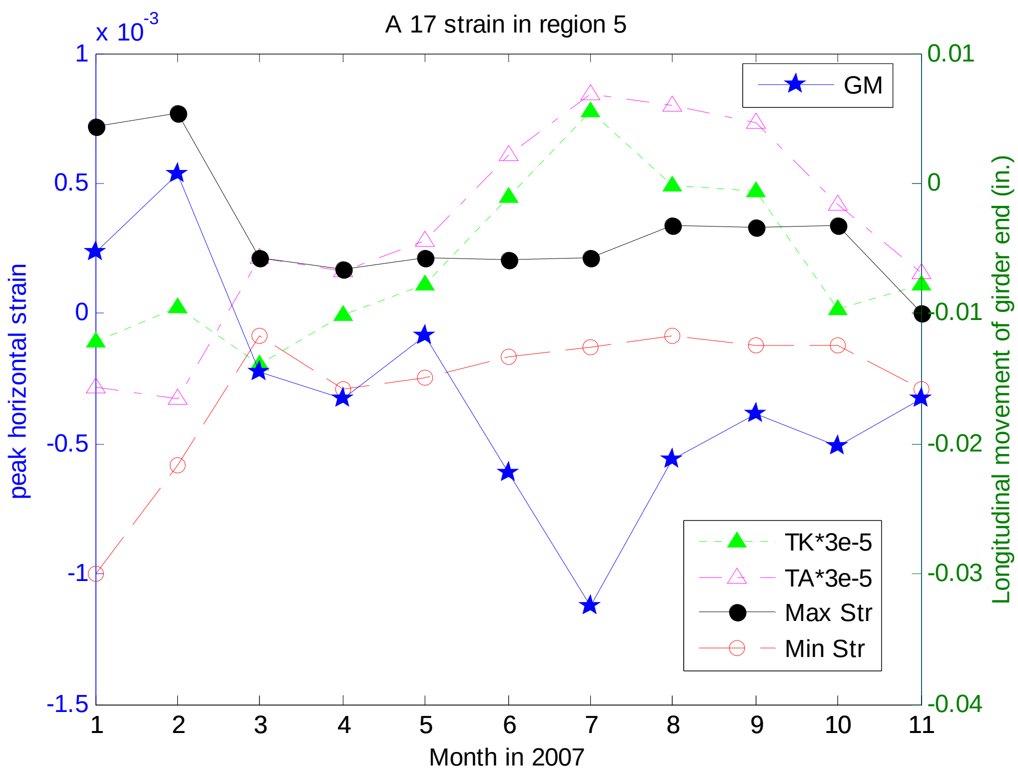


Figure C-93 Variation of peak strain in region 5 of Bridge A 1.7 with time and temperature

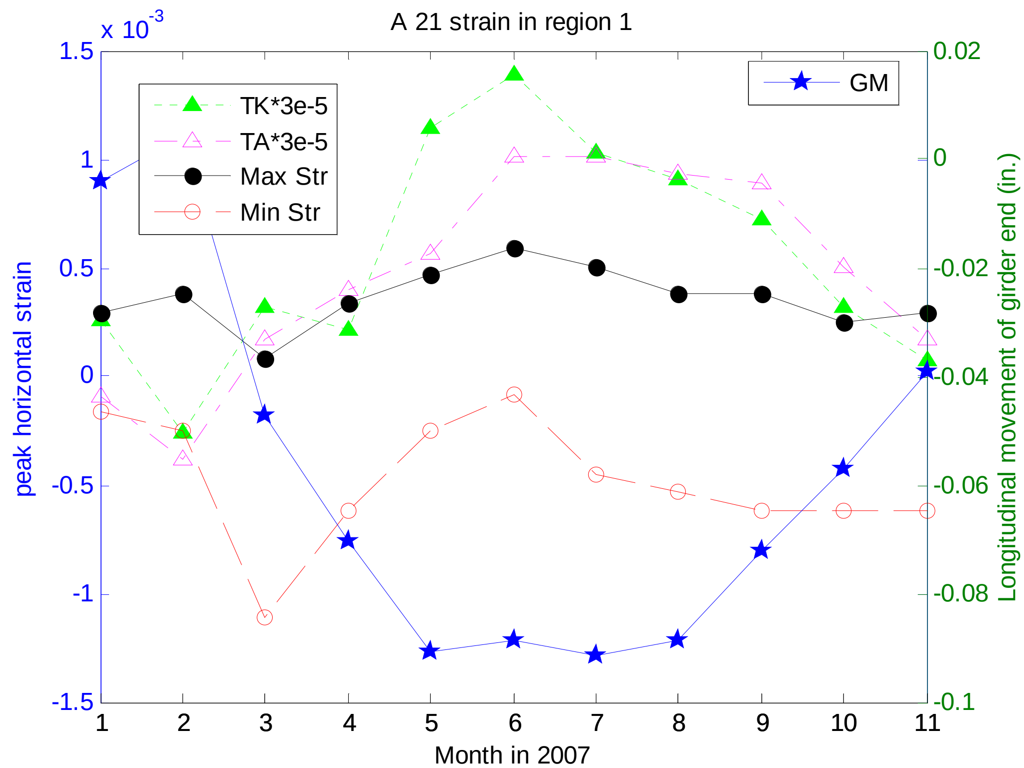


Figure C-94 Variation of peak strain in region 1 of Bridge A 2.1 with time and temperature

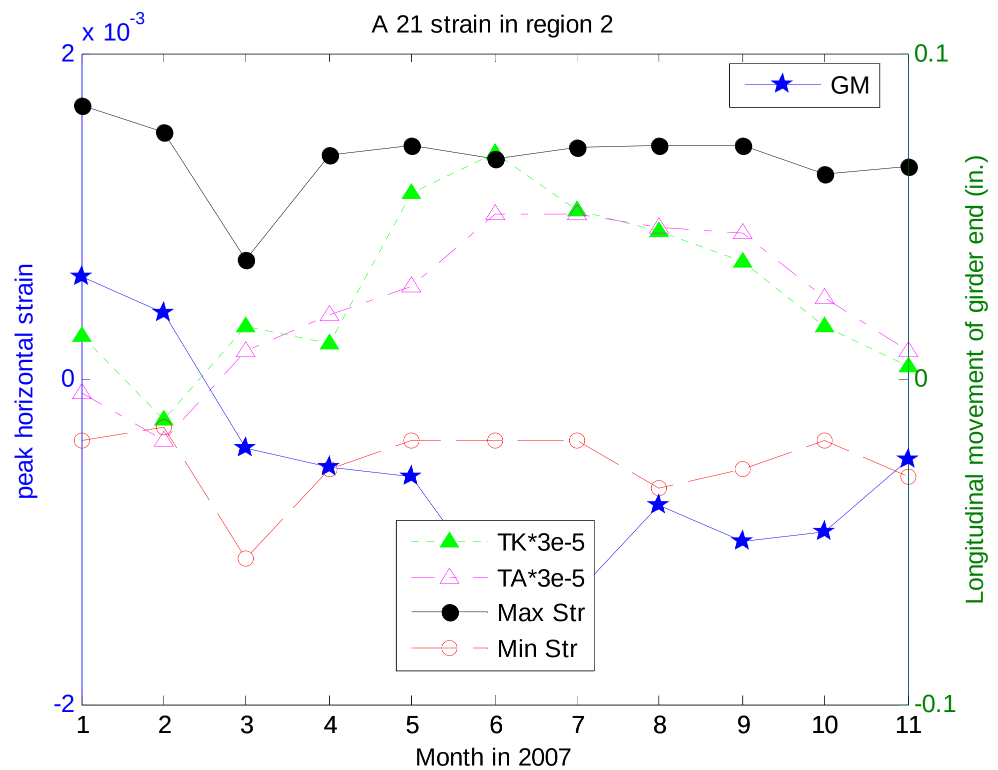


Figure C-95 Variation of peak strain in region 2 of Bridge A 2.1 with time and temperature

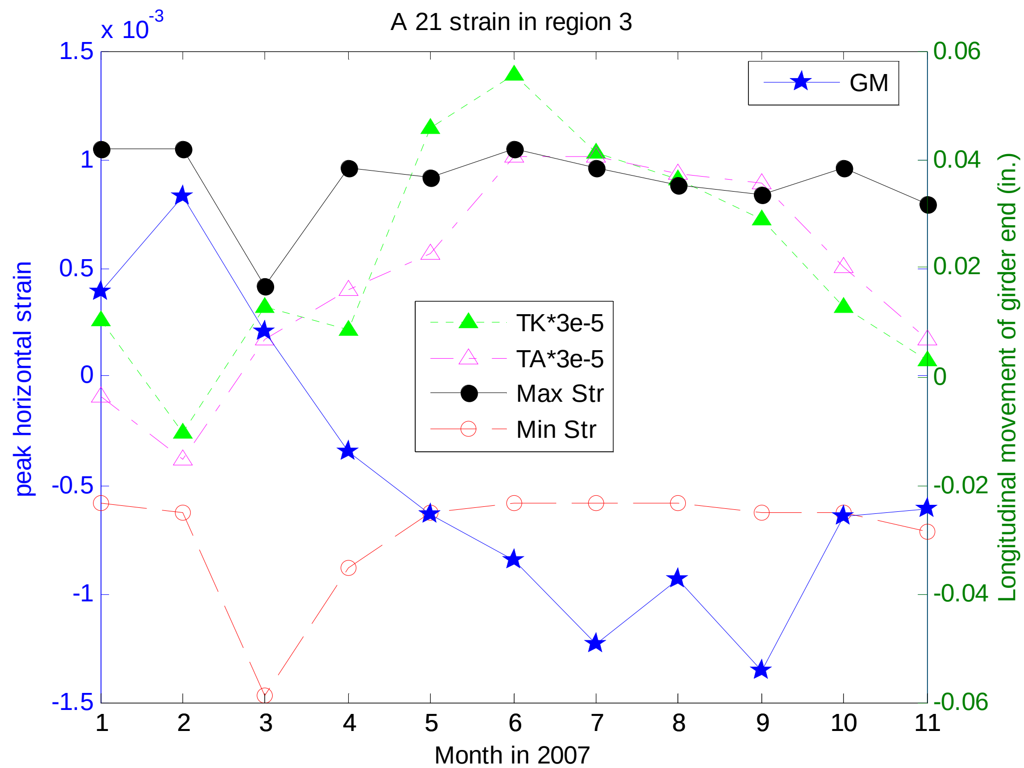


Figure C-96 Variation of peak strain in region 3 of Bridge A 2.1 with time and temperature

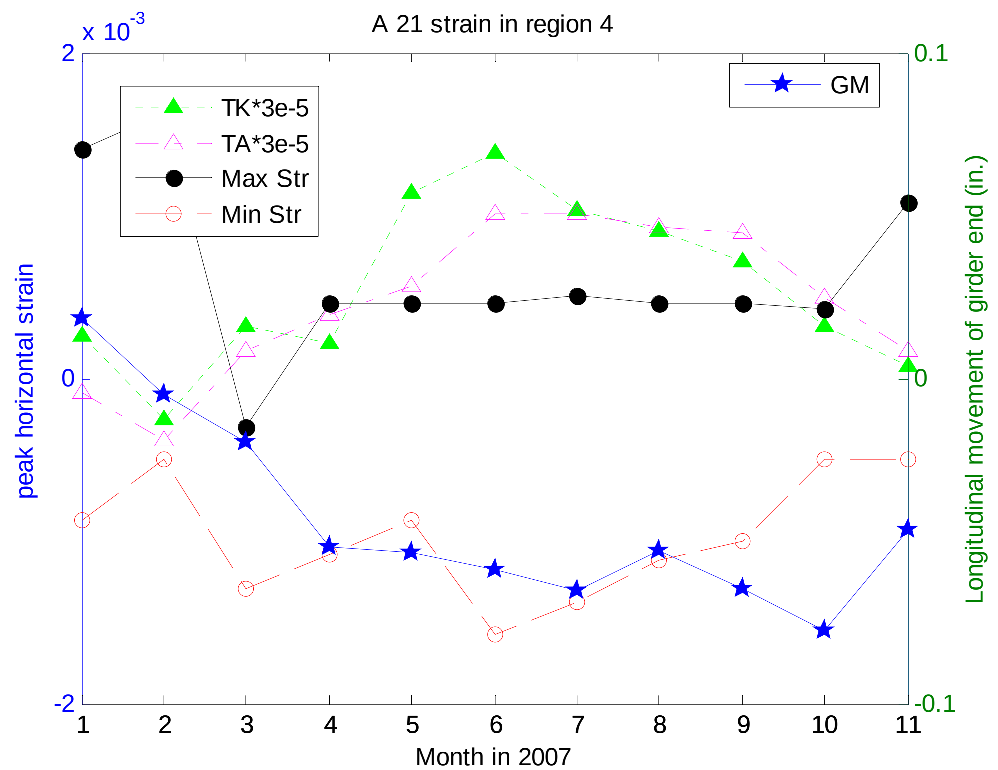


Figure C-97 Variation of peak strain in region 4 of Bridge A 2.1 with time and temperature

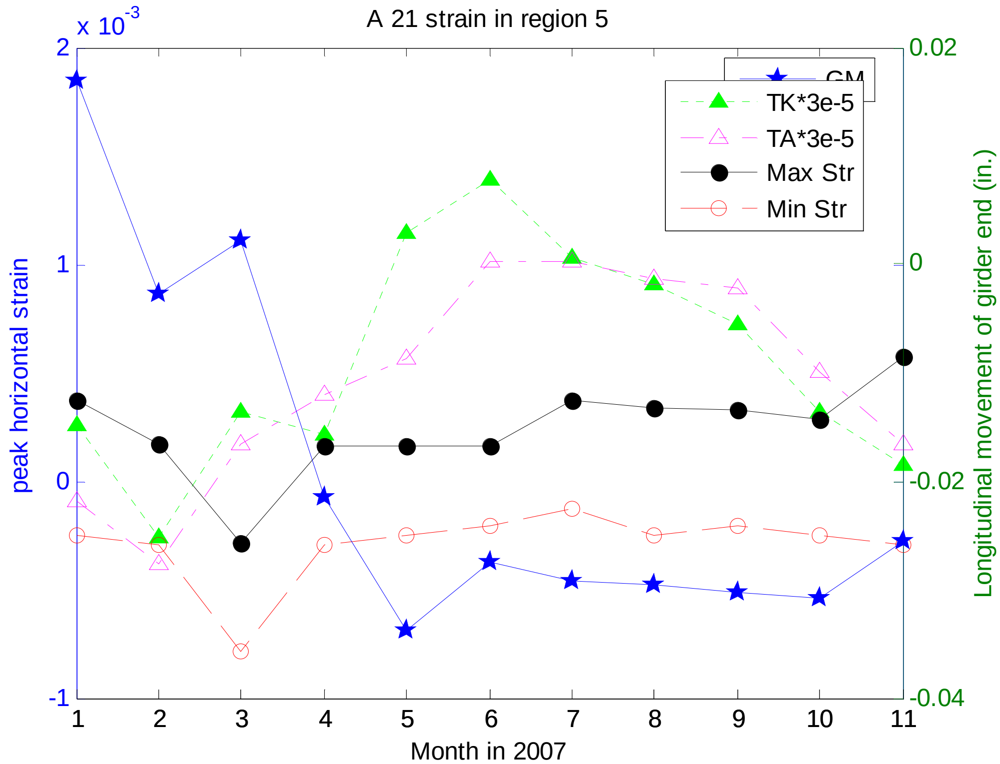


Figure C-98 Variation of peak strain in region 5 of Bridge A 2.1 with time and temperature

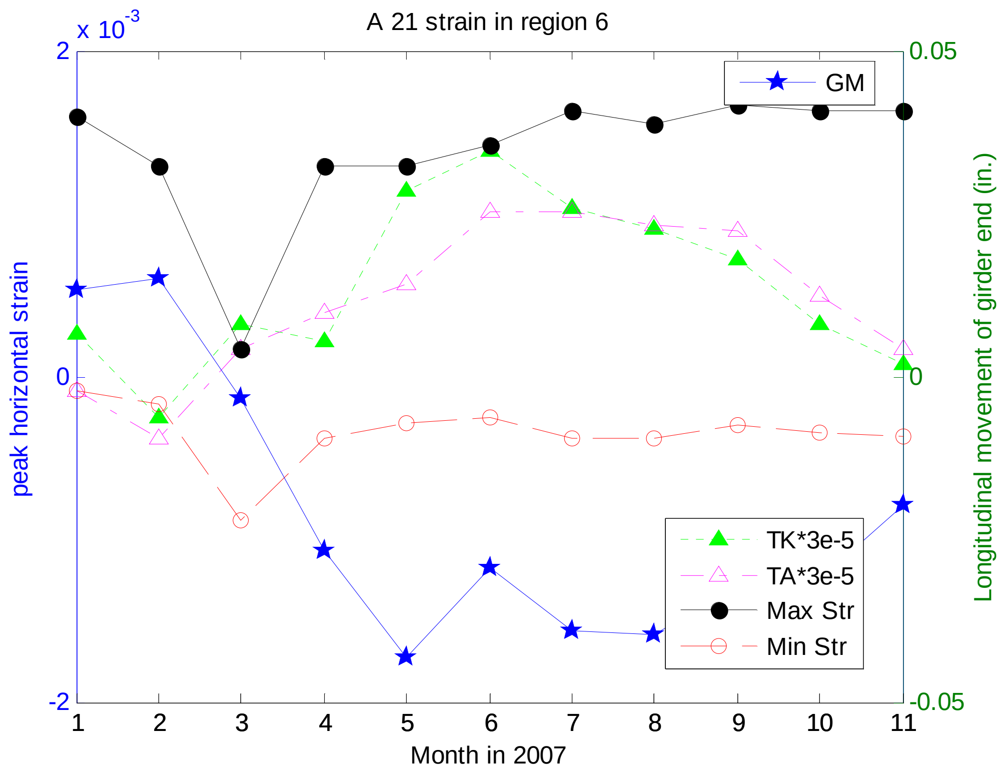


Figure C-99 Variation of peak strain in region 6 of Bridge A 2.1 with time and temperature

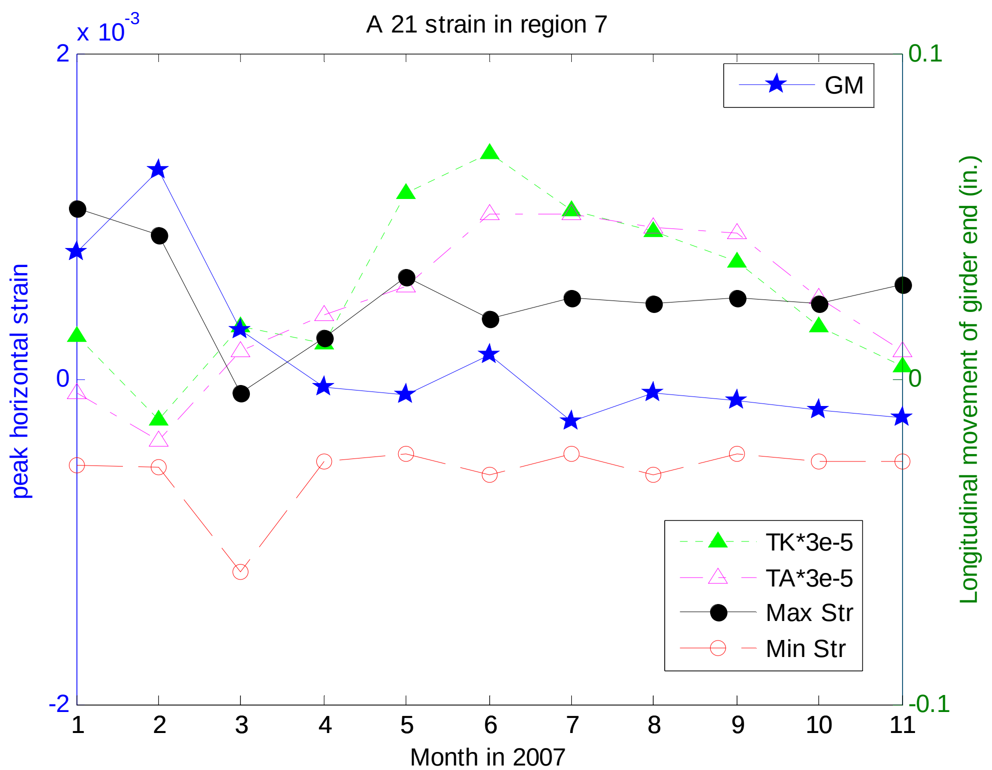


Figure C-100 Variation of peak strain in region 7 of Bridge A 2.1 with time and temperature

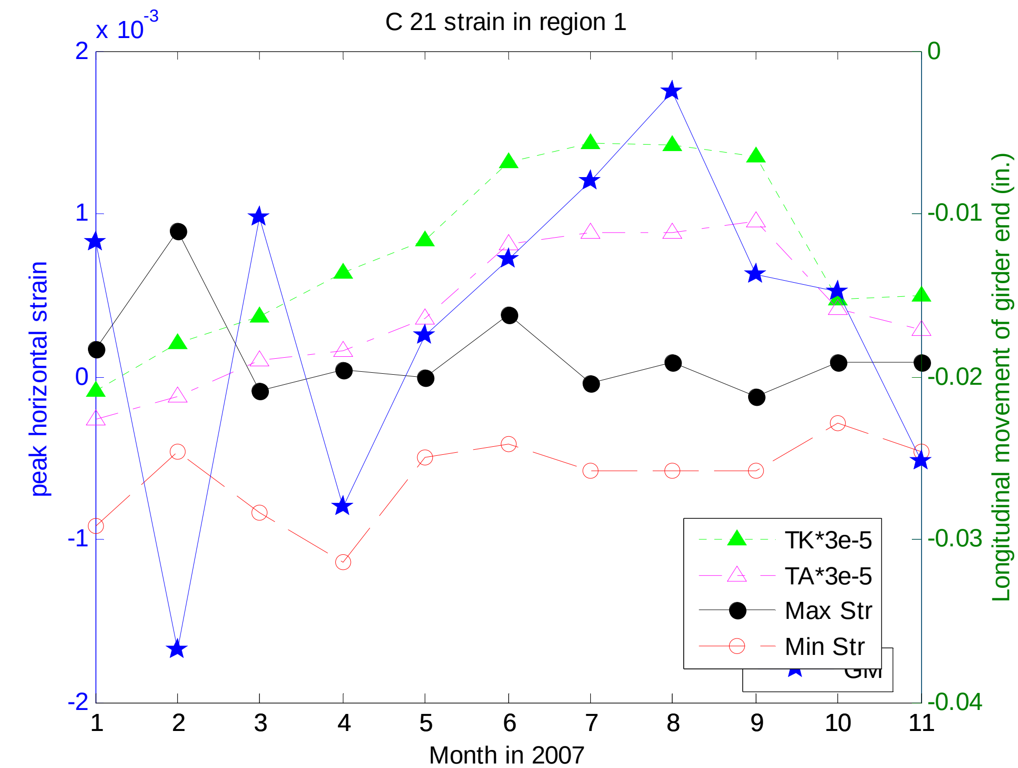


Figure C-101 Variation of peak strain in region 1 of Bridge C 2.1 with time and temperature

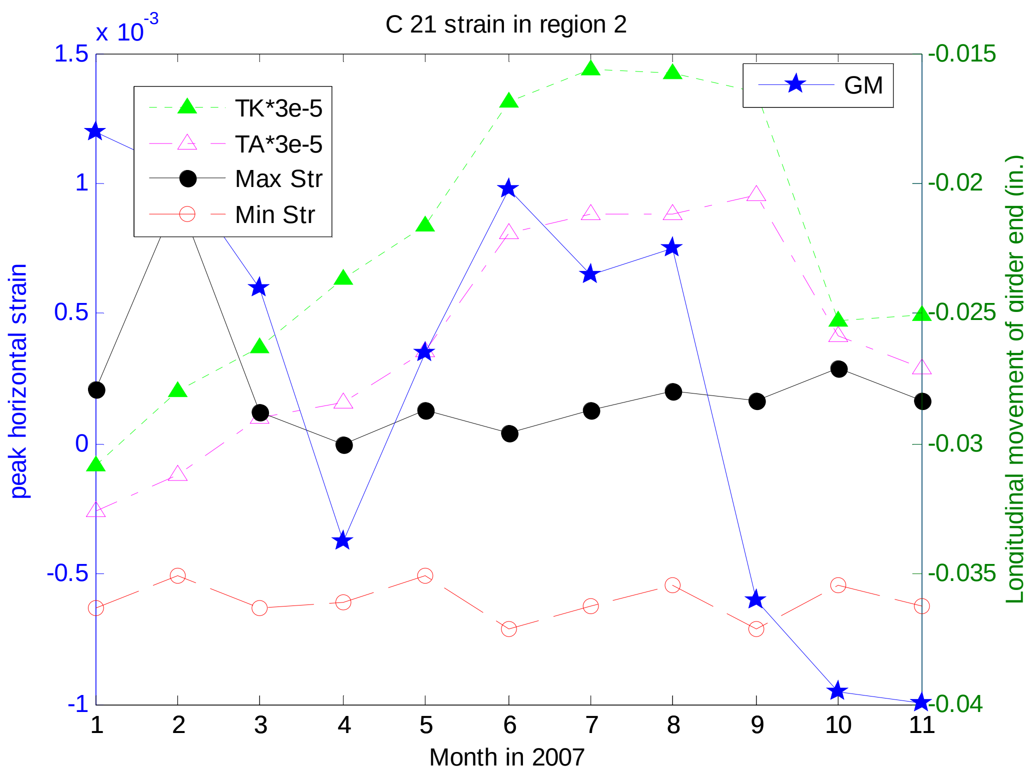


Figure C-102 Variation of peak strain in region 2 of Bridge C 2.1 with time and temperature

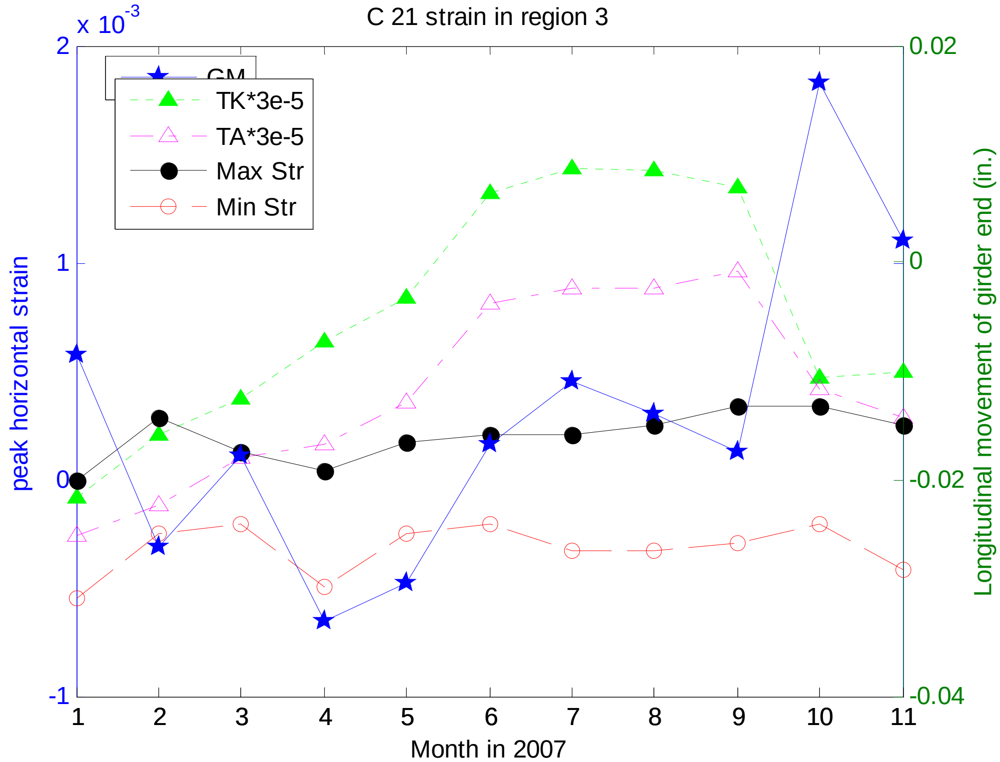


Figure C-103 Variation of peak strain in region 3 of Bridge C 2.1 with time and temperature

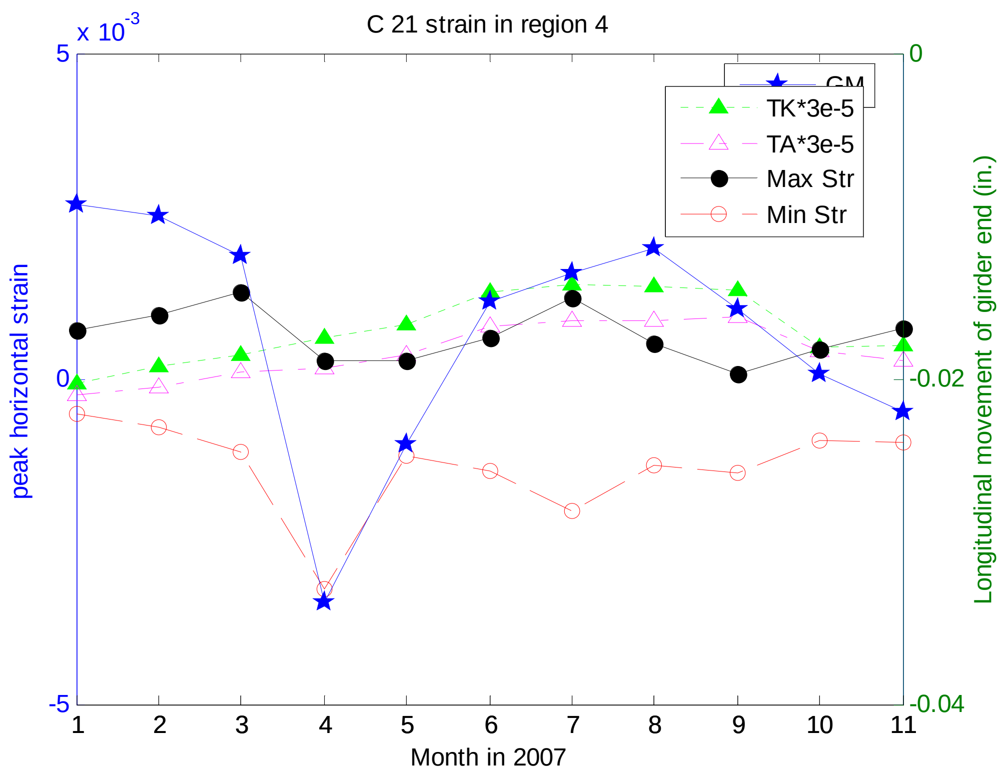


Figure C-104 Variation of peak strain in region 4 of Bridge C 2.1 with time and temperature

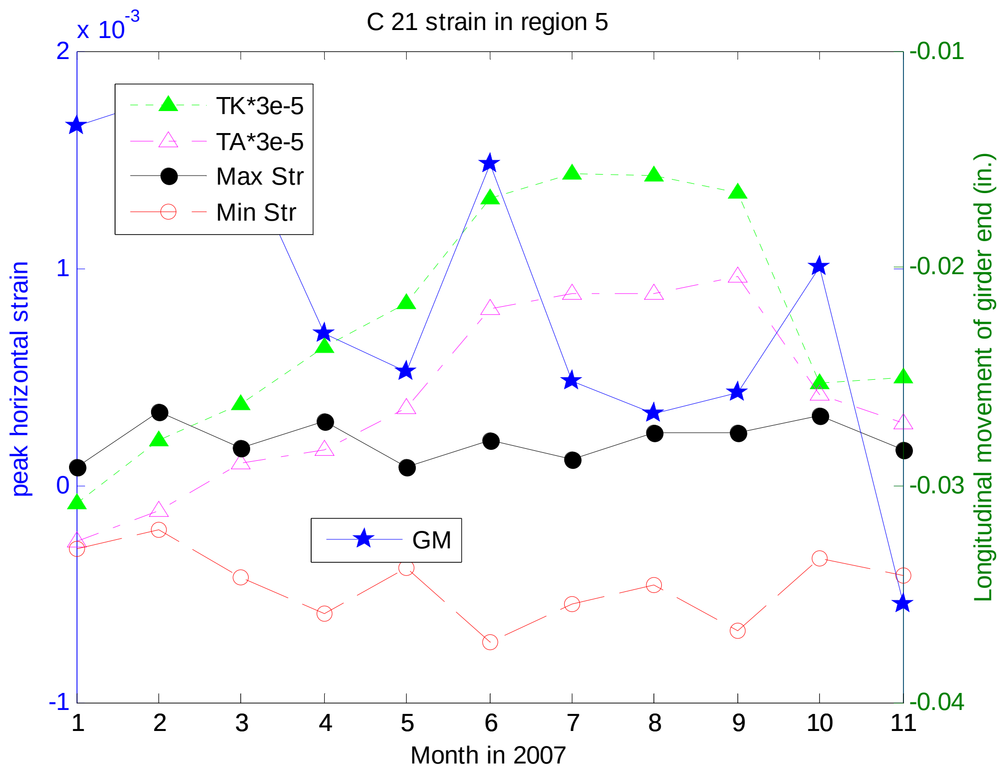


Figure C-105 Variation of peak strain in region 5 of Bridge C 2.1 with time and temperature

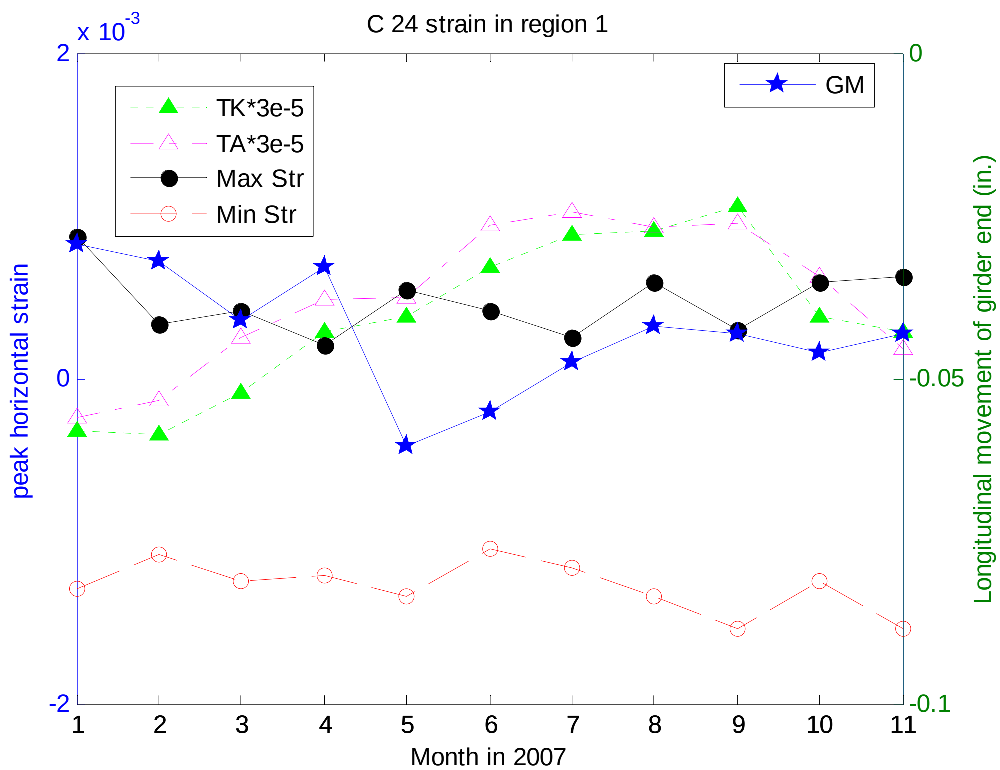


Figure C-106 Variation of peak strain in region 1 of Bridge C 2.4 with time and temperature

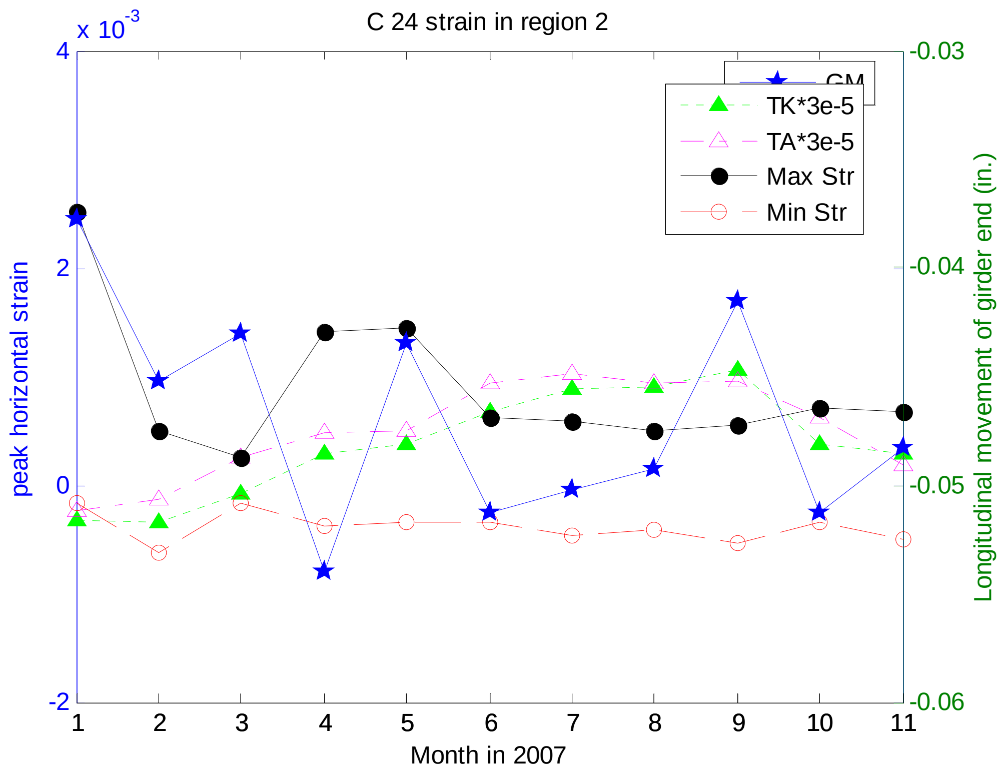


Figure C-107 Variation of peak strain in region 2 of Bridge C 2.4 with time and temperature

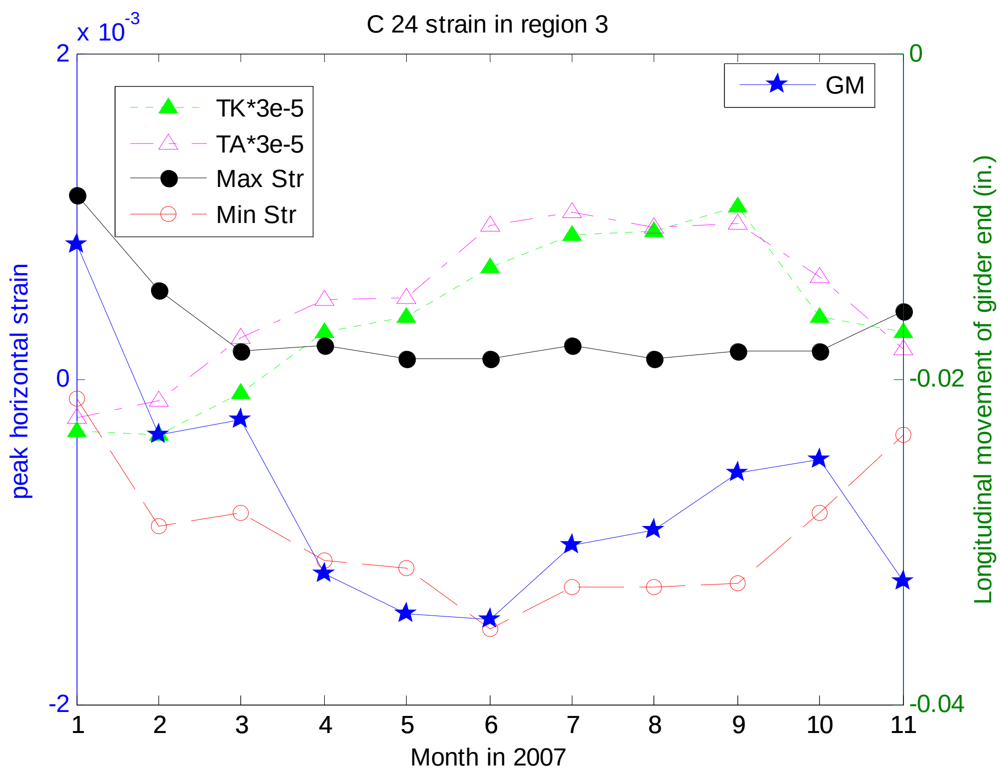


Figure C-108 Variation of peak strain in region 3 of Bridge C 2.4 with time and temperature

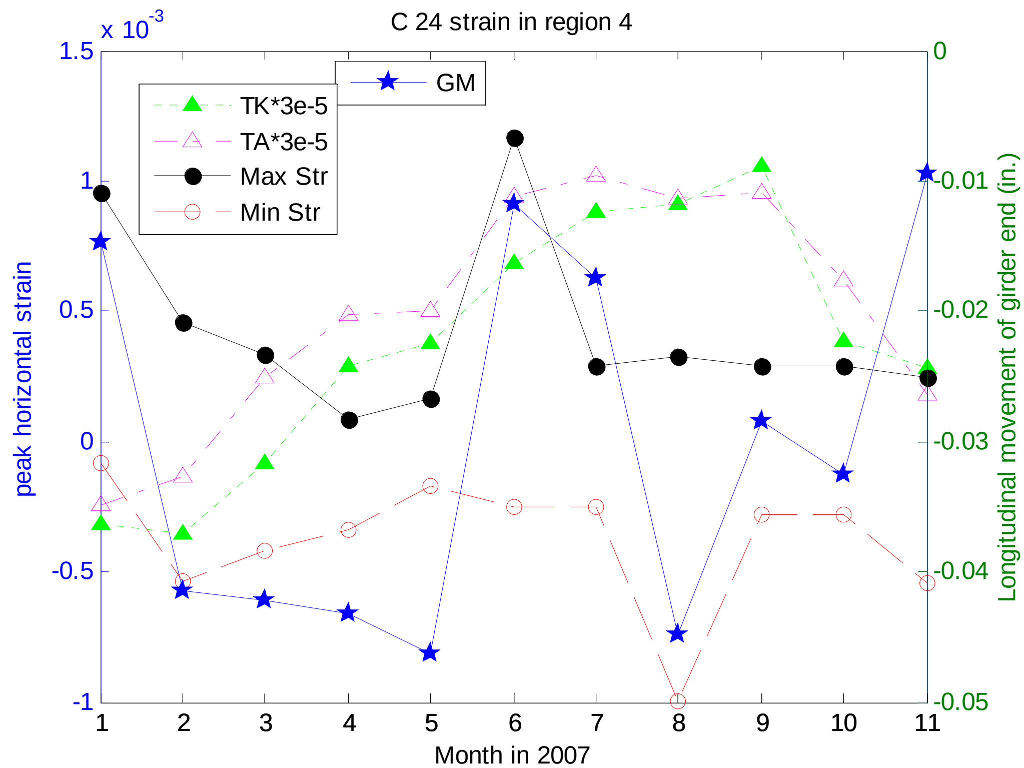


Figure C-109 Variation of peak strain in region 4 of Bridge C 2.4 with time and temperature

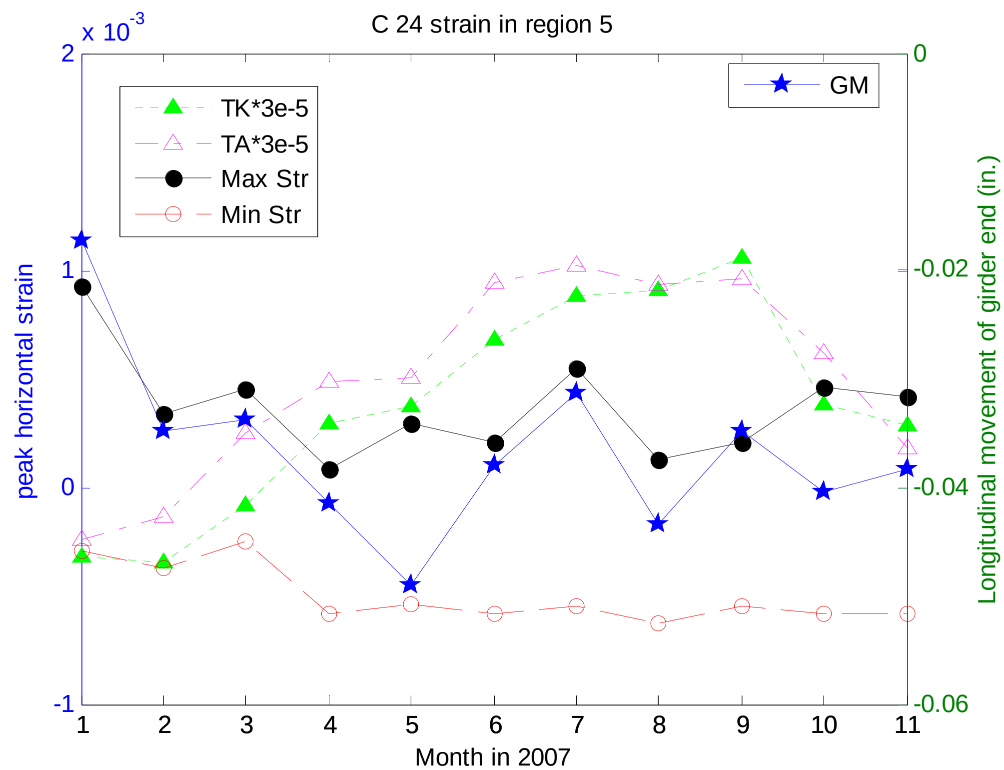


Figure C-110 Variation of peak strain in region 5 of Bridge C 2.4 with time and temperature

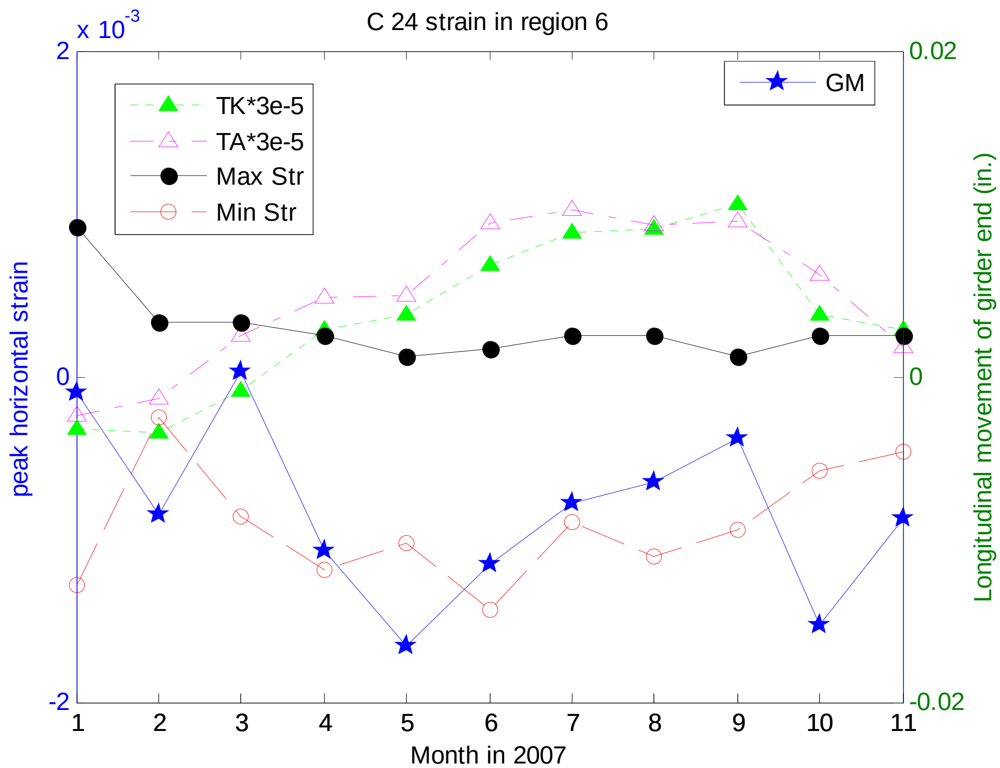


Figure C-111 Variation of peak strain in region 6 of Bridge C 2.4 with time and temperature

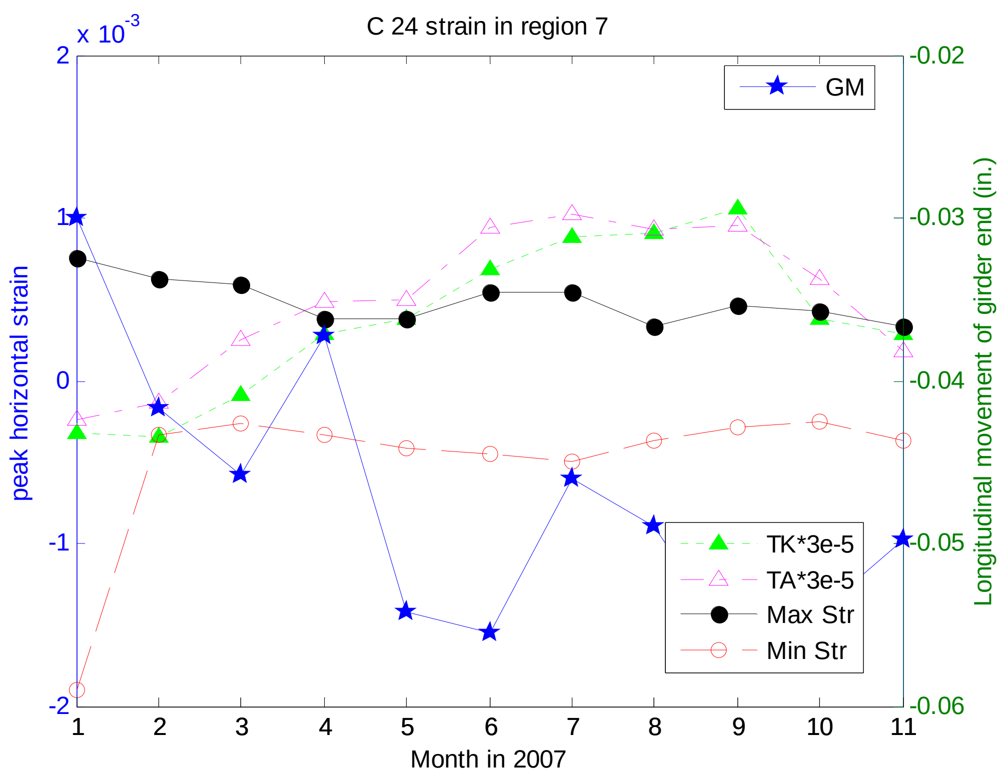


Figure C-112 Variation of peak strain in region 7 of Bridge C 2.4 with time and temperature

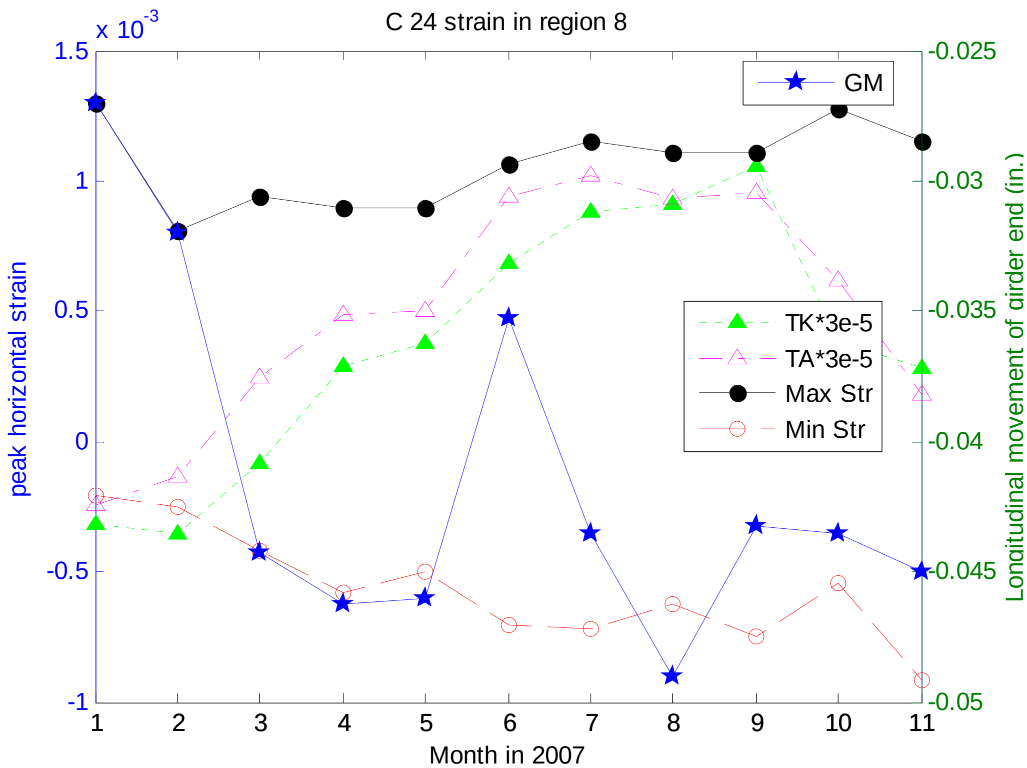


Figure C-113 Variation of peak strain in region 8 of Bridge C 2.4 with time and temperature

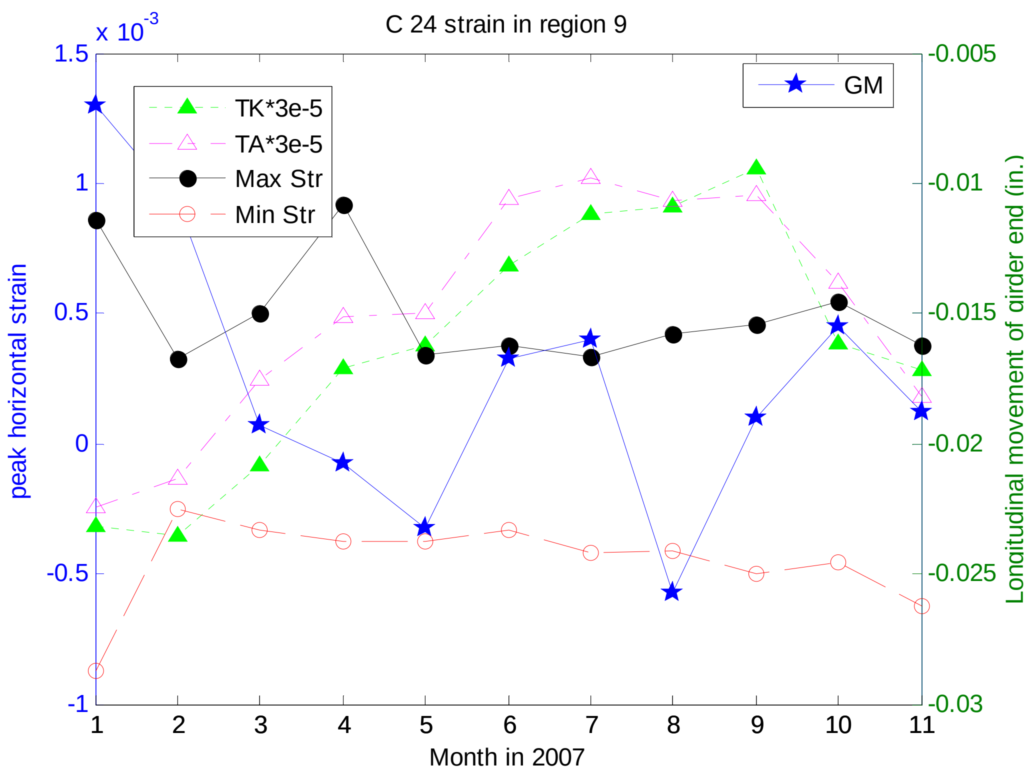


Figure C-114 Variation of peak strain in region 9 of Bridge C 2.4 with time and temperature

C.III Peak Strain vs. Time and Temperature in Region between Girders

Variation of maximum and minimum strain in the spacing in abutment wall of A 1.7 is plotted in Figure C-115 to **Figure** C-119. Similarly, variation of maximum and minimum strain in the regions in abutment wall of A 2.1, C 2.1, and C 2.4 is plotted in **Figure** C-120 to **Figure** C-126, **Figure** C-127 to **Figure** C-131, and **Figure** C-132 to **Figure** C-136; respectively.

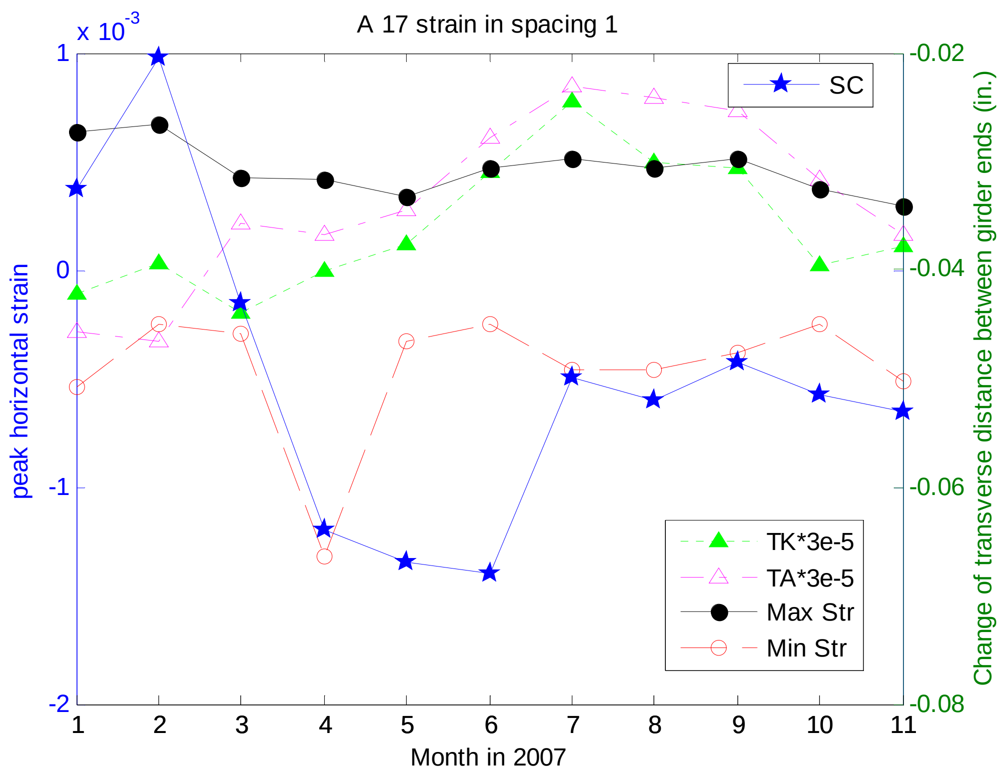


Figure C-115 Variation of peak strain in spacing 1 of Bridge A 1.7 with time and temperature

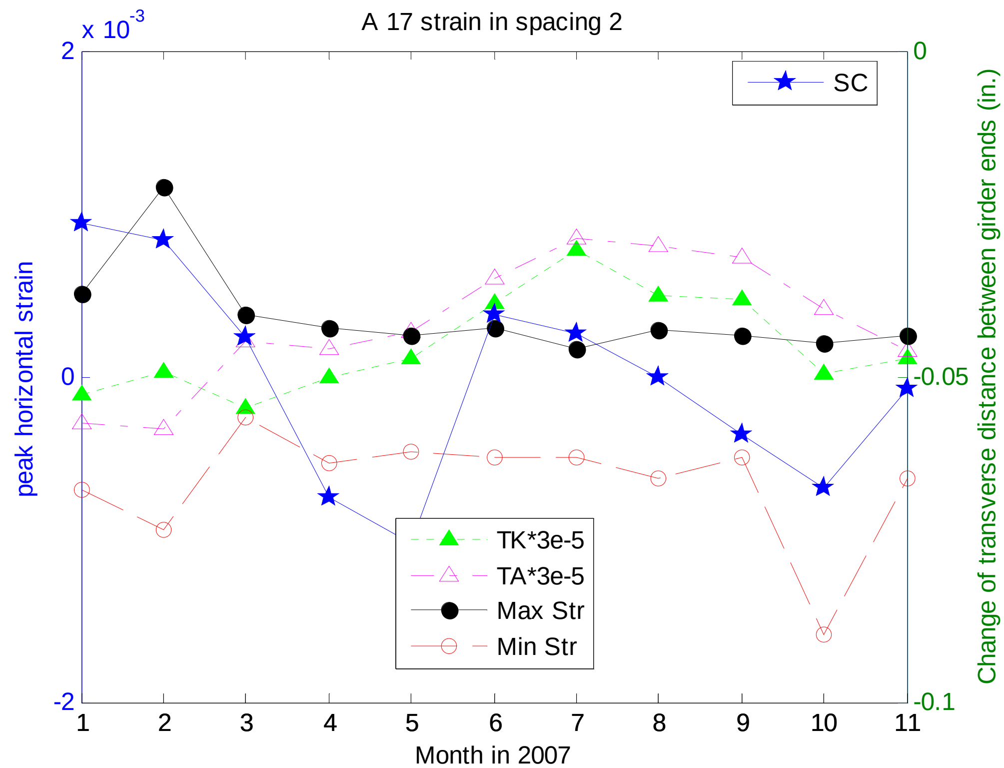


Figure C-116 Variation of peak strain in spacing 2 of Bridge A 1.7 with time and temperature

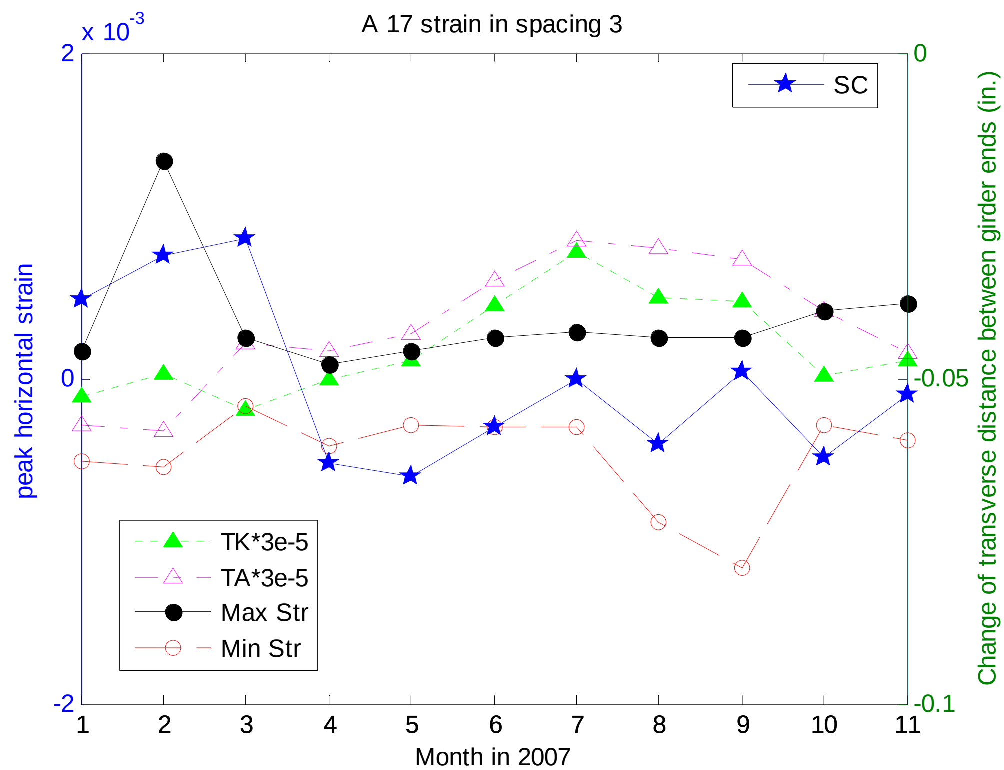


Figure C-117 Variation of peak strain in spacing 3 of Bridge A 1.7 with time and temperature

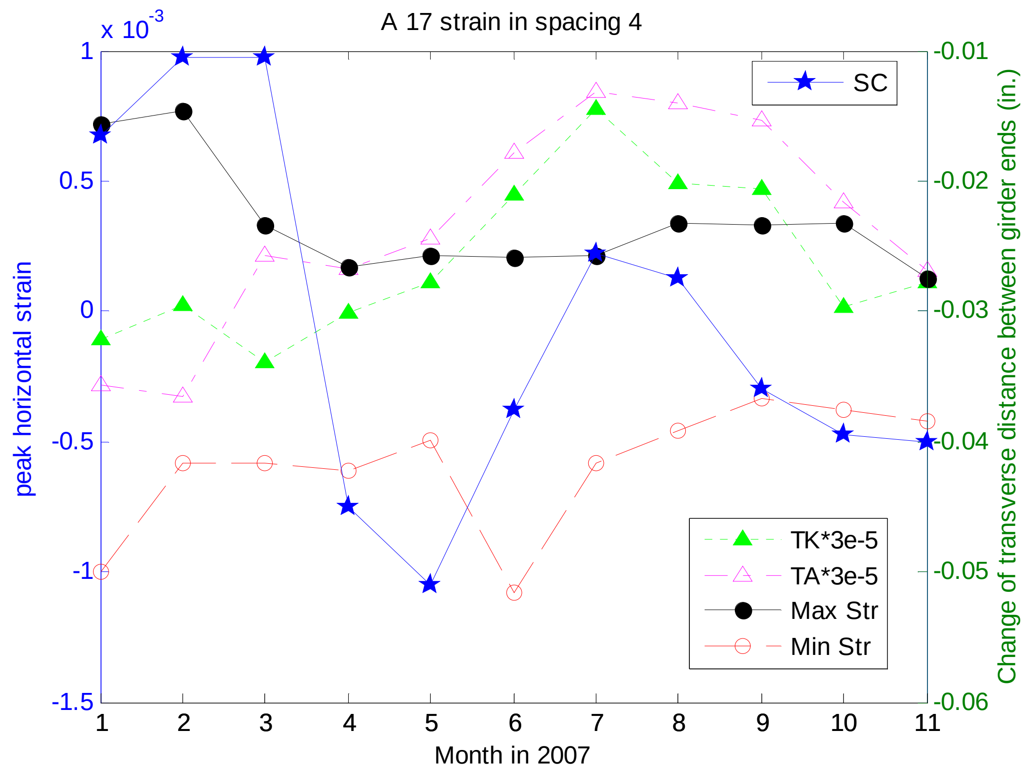


Figure C-118 Variation of peak strain in spacing 4 of Bridge A 1.7 with time and temperature

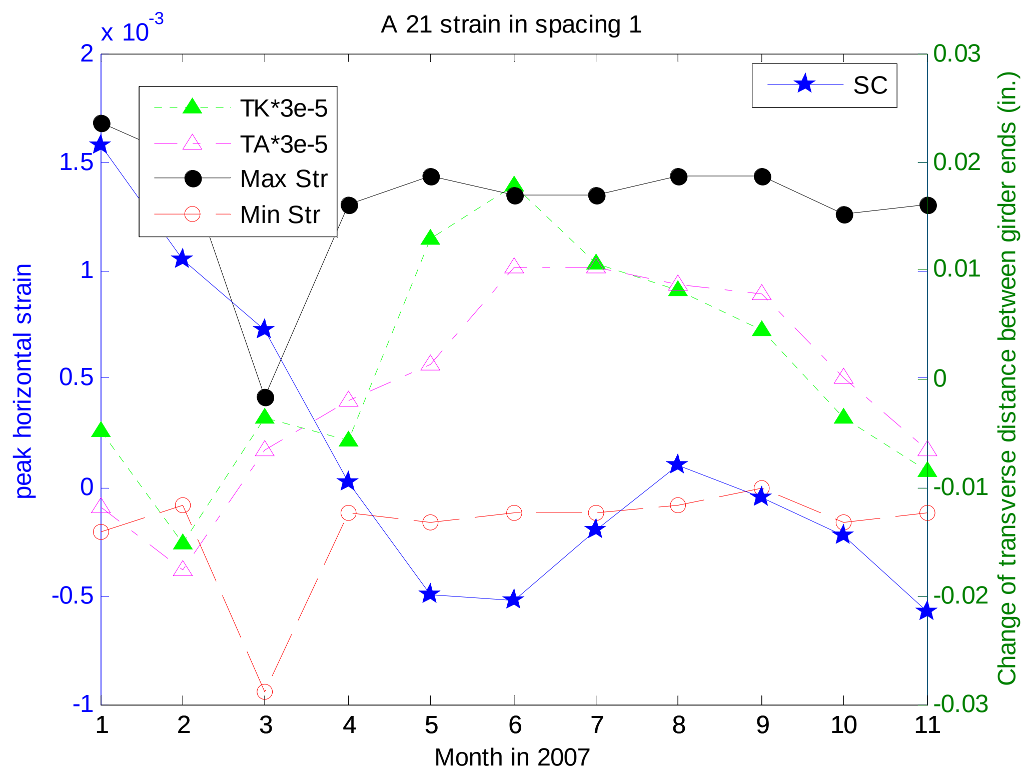


Figure C-119 Variation of peak strain in spacing 1 of Bridge A 2.1 with time and temperature

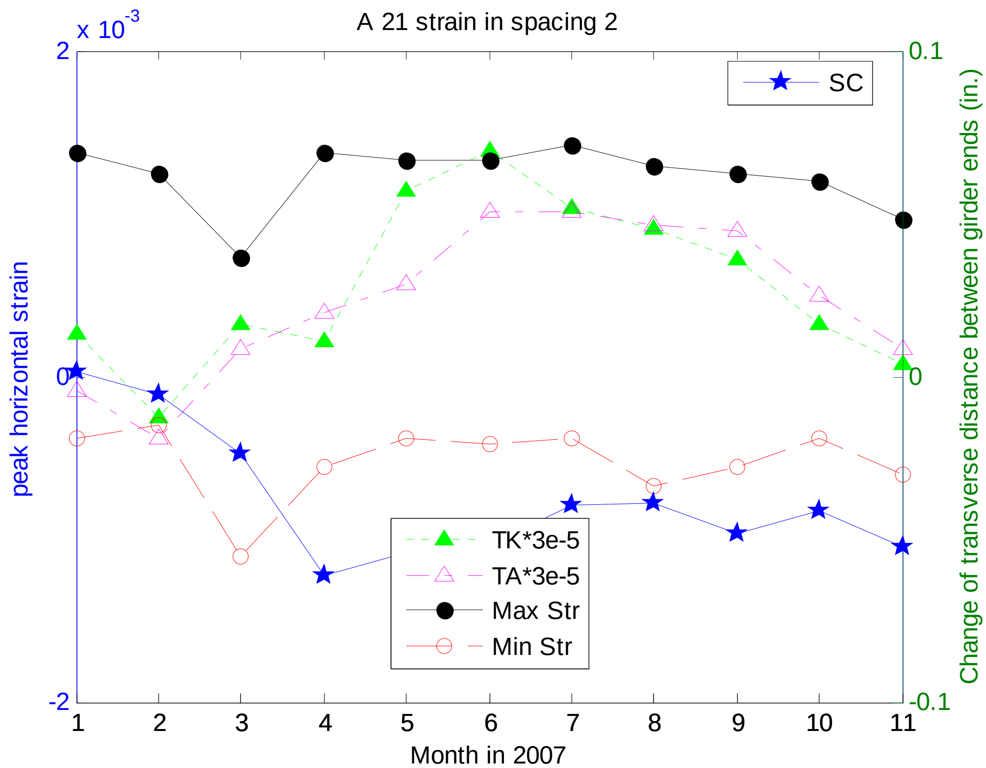


Figure C-120 Variation of peak strain in spacing 2 of Bridge A 2.1 with time and temperature

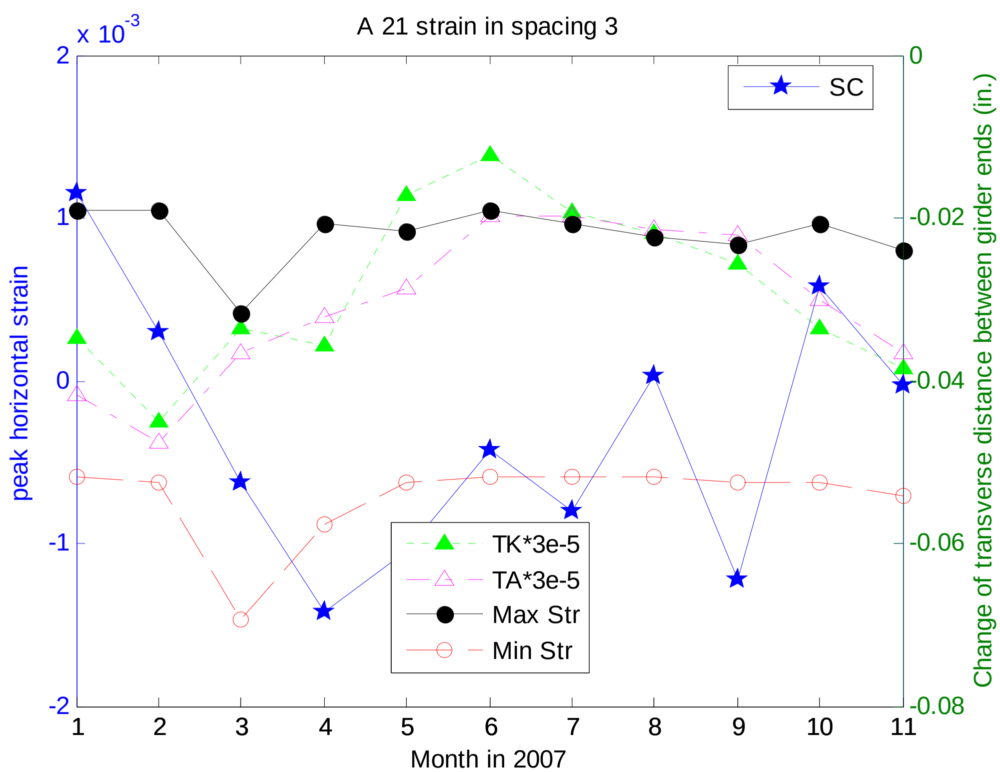


Figure C-121 Variation of peak strain in spacing 3 of Bridge A 2.1 with time and temperature

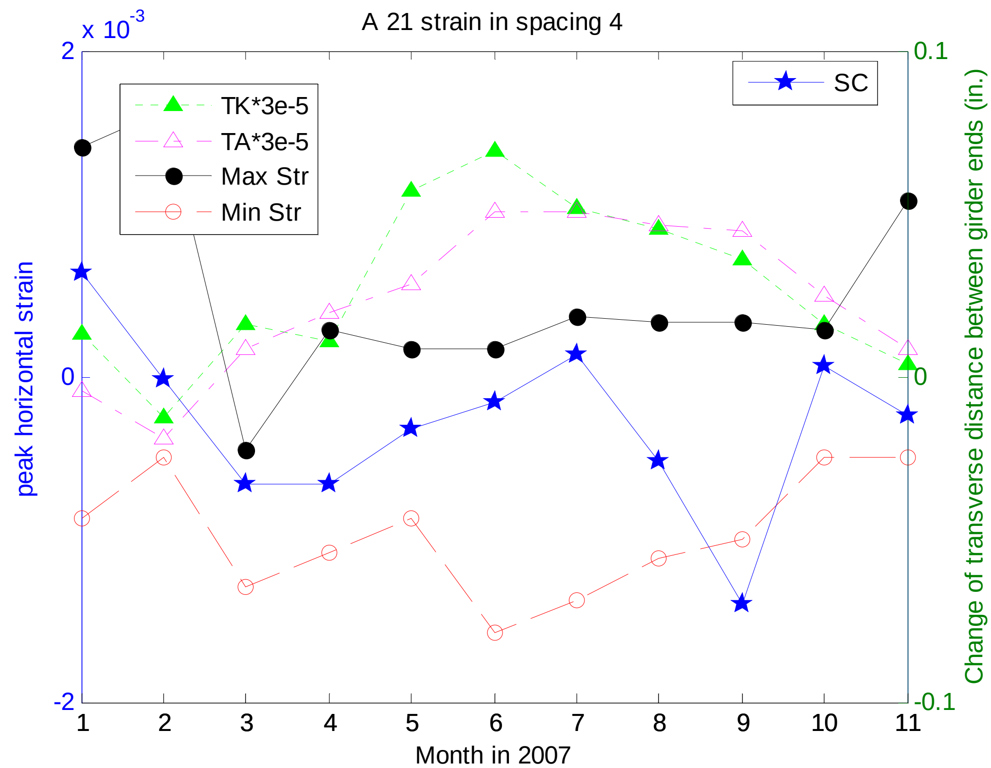


Figure C-122 Variation of peak strain in spacing 4 of Bridge A 2.1 with time and temperature

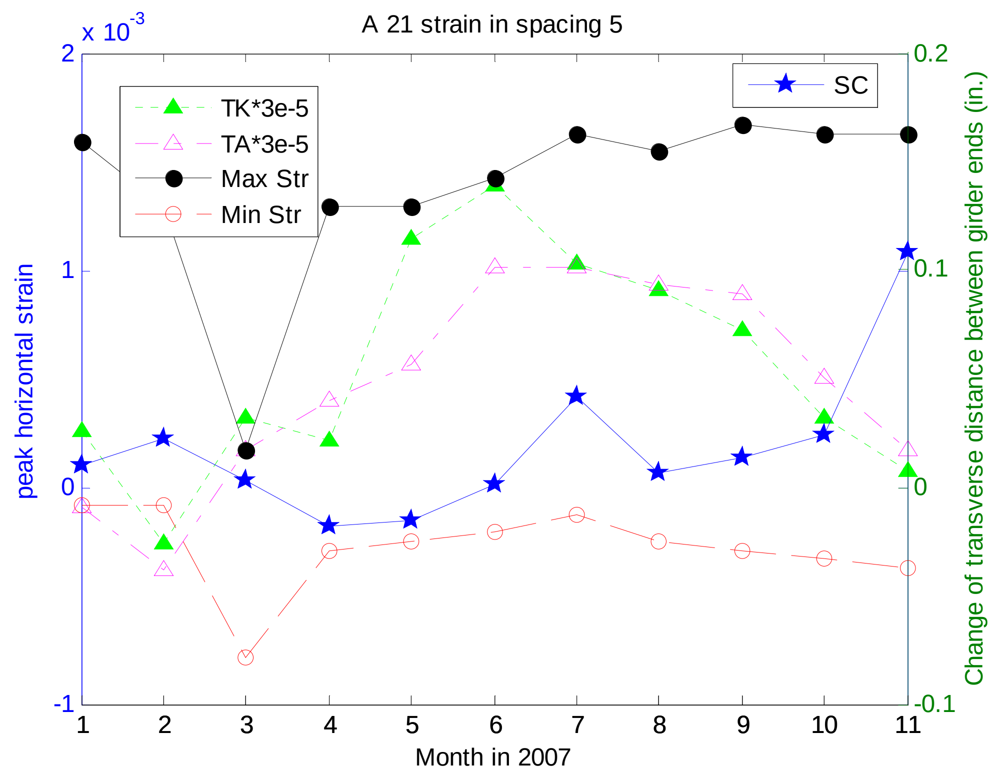


Figure C-123 Variation of peak strain in spacing 5 of Bridge A 2.1 with time and temperature

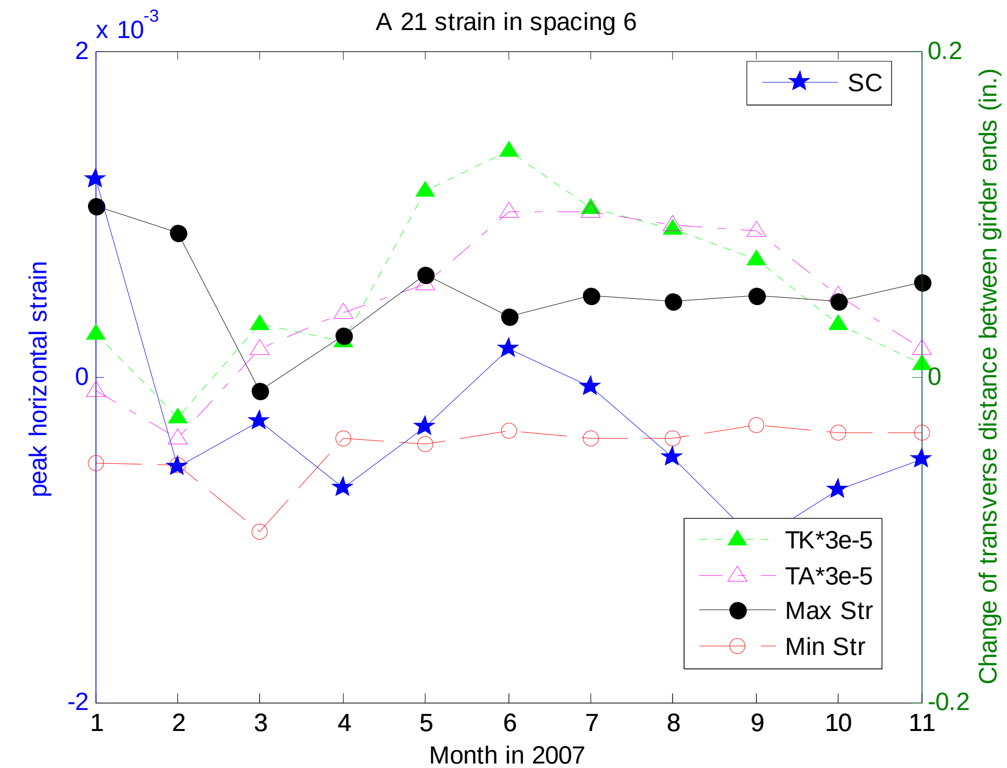


Figure C-124 Variation of peak strain in spacing 6 of Bridge A 2.1 with time and temperature

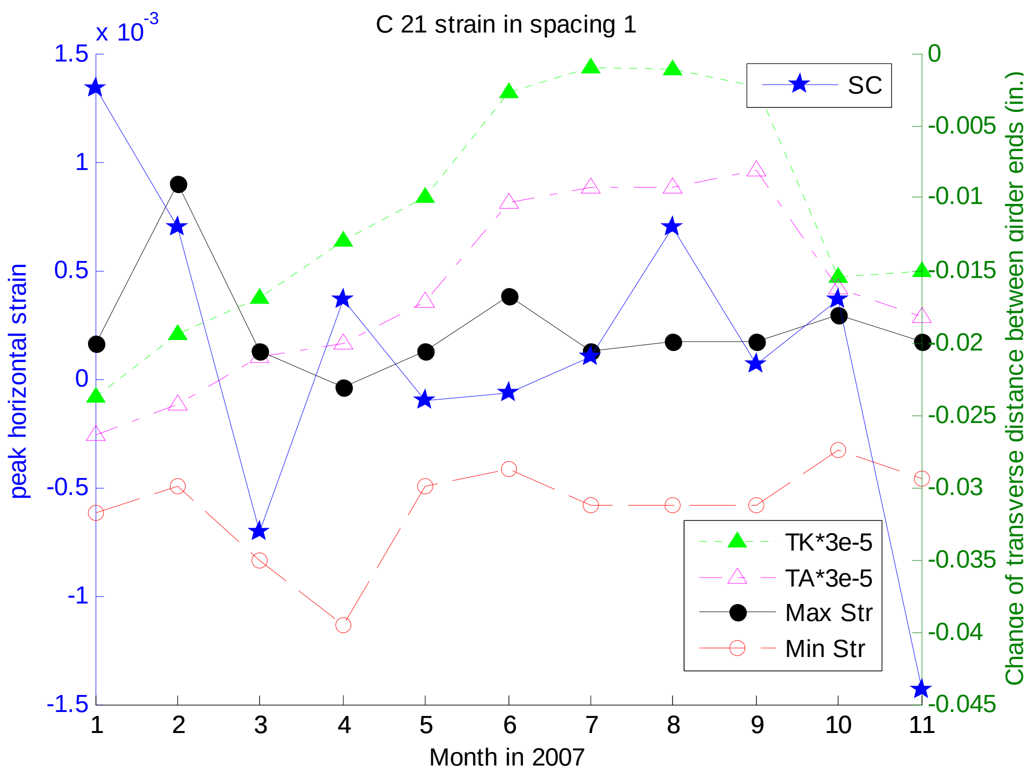


Figure C-125 Variation of peak strain in spacing 1 of Bridge C 2.1 with time and temperature

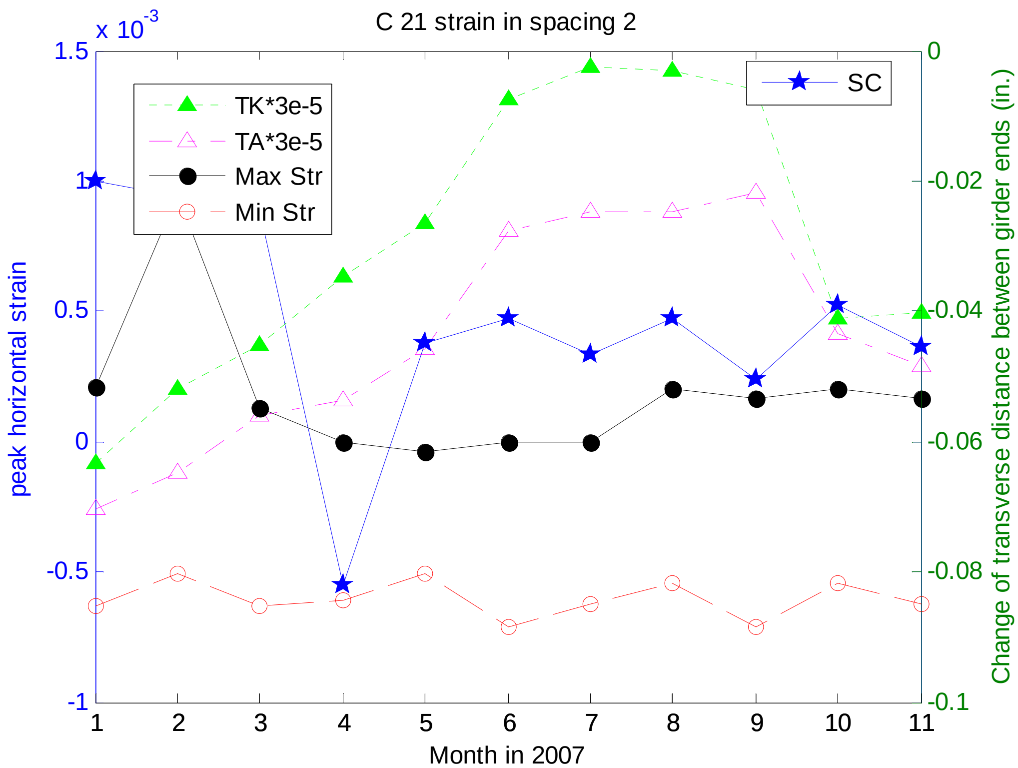


Figure C-126 Variation of peak strain in spacing 2 of Bridge C 2.1 with time and temperature

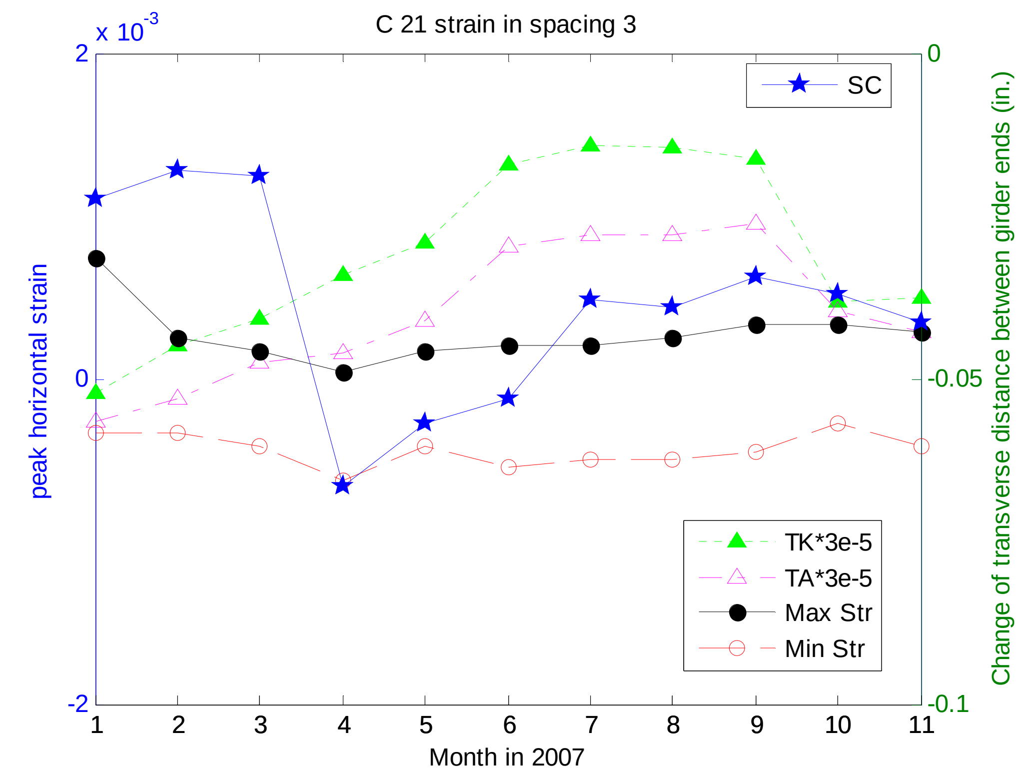


Figure C-127 Variation of peak strain in spacing 3 of Bridge C 2.1 with time and temperature

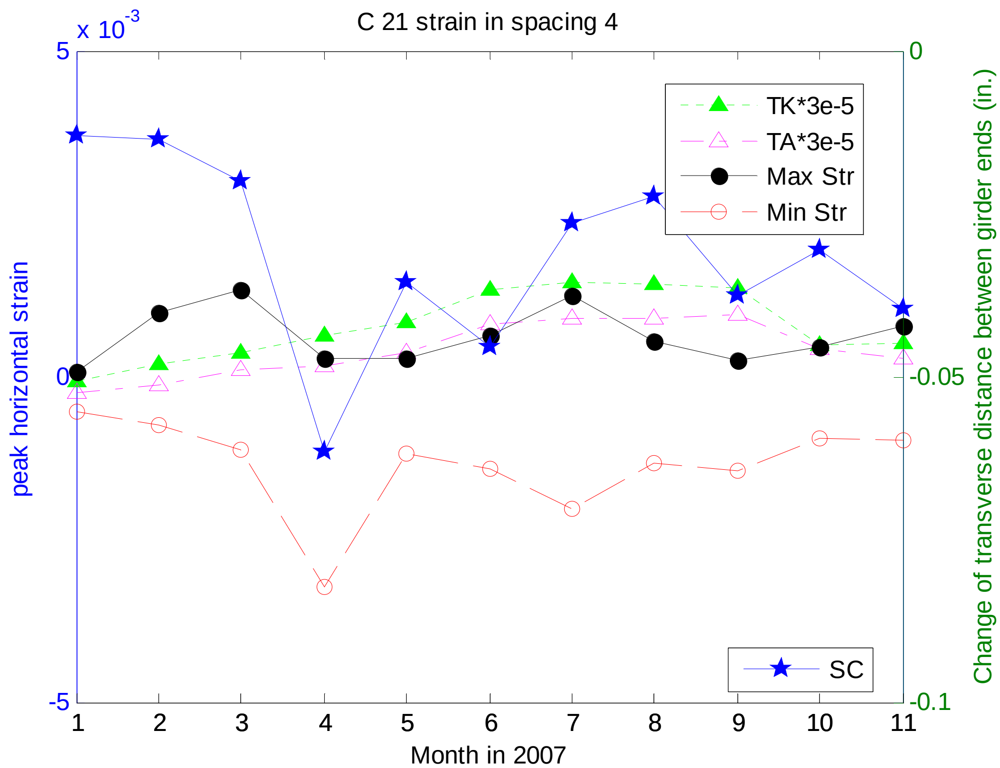


Figure C-128 Variation of peak strain in spacing 4 of Bridge C 2.1 with time and temperature

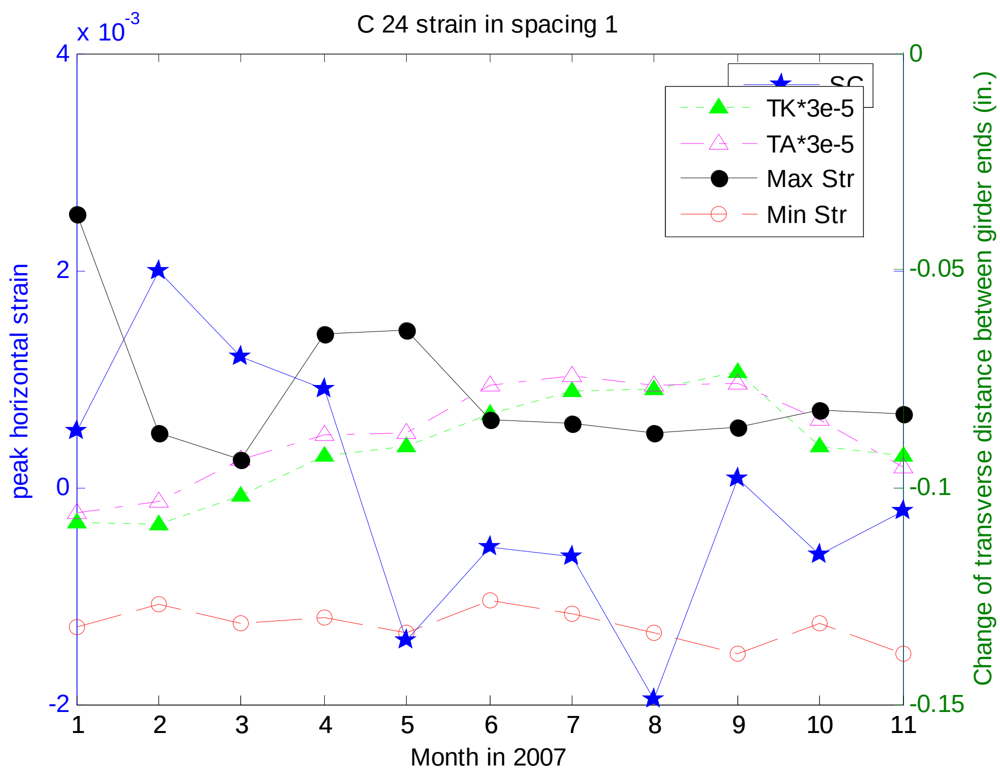


Figure C-129 Variation of peak strain in spacing 1 of Bridge C 2.4 with time and temperature

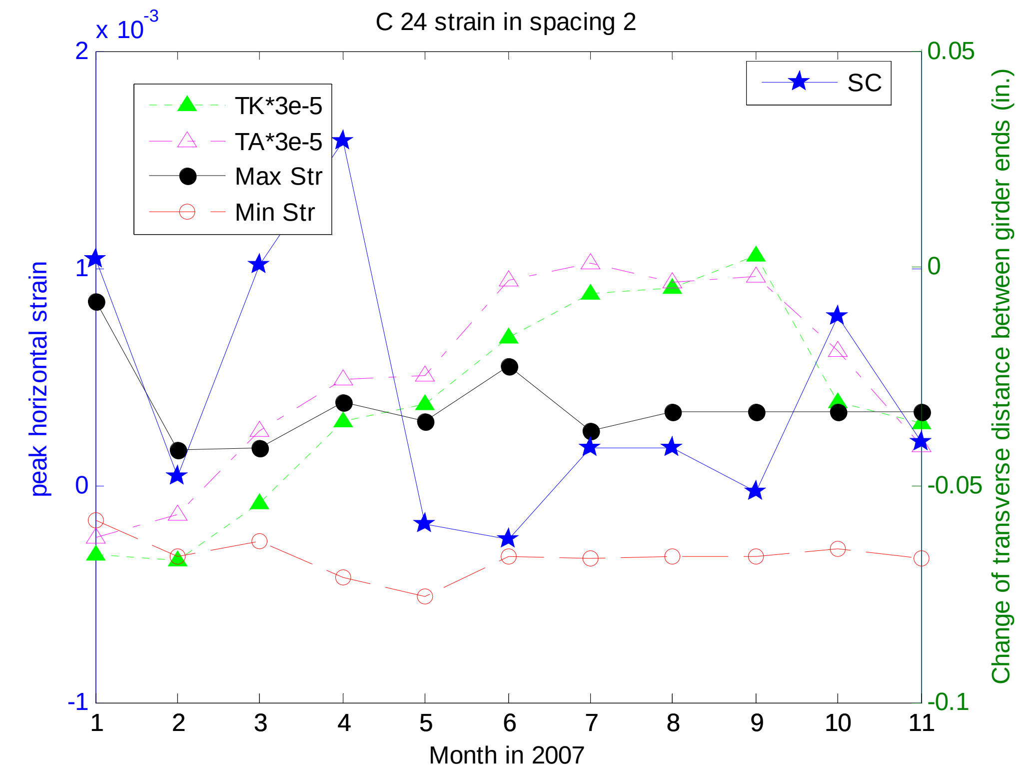


Figure C-130 Variation of peak strain in spacing 2 of Bridge C 2.4 with time and temperature

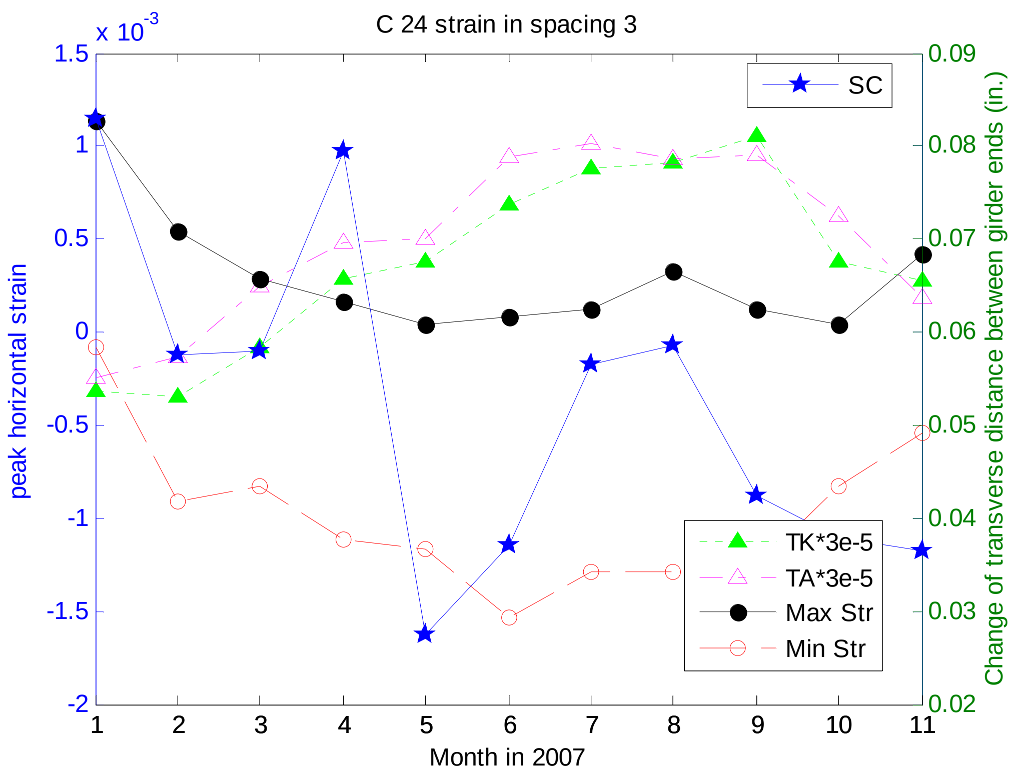


Figure C-131 Variation of peak strain in spacing 3 of Bridge C 2.4 with time and temperature

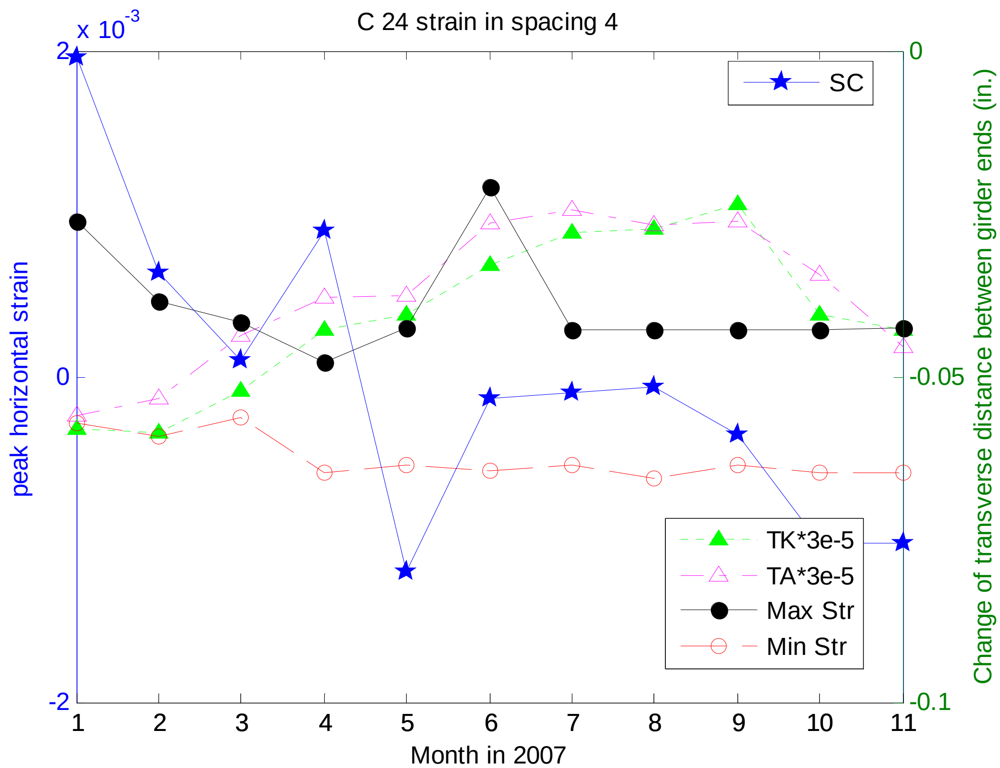


Figure C-132 Variation of peak strain in spacing 4 of Bridge C 2.1 with time and temperature

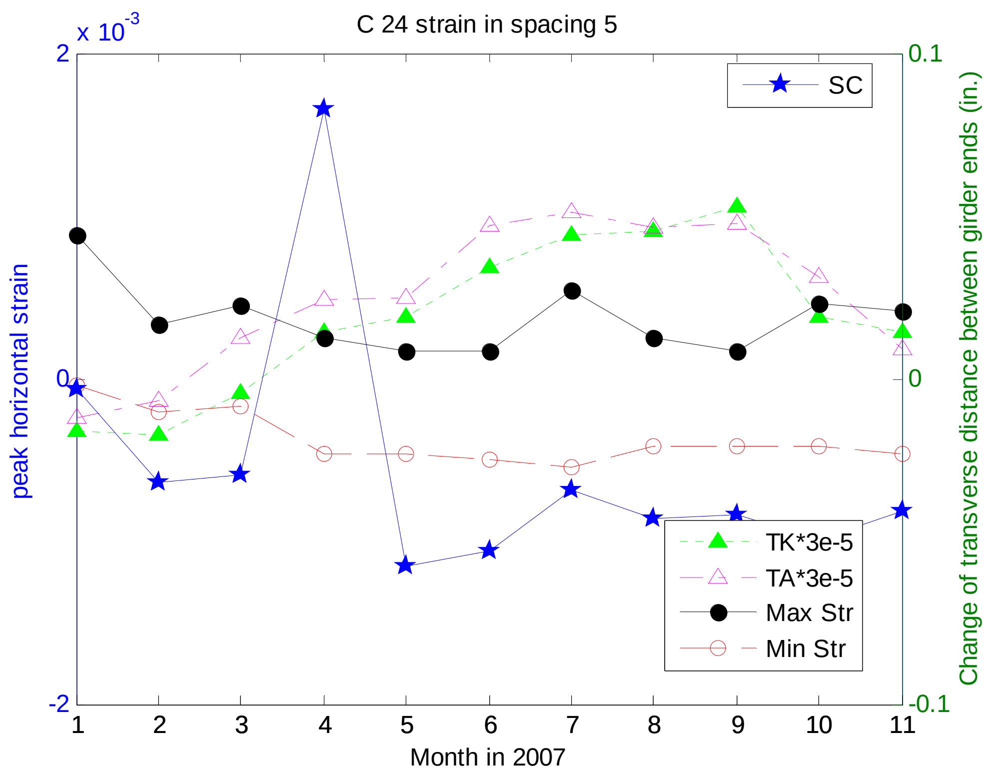


Figure C-133 Variation of peak strain in spacing 5 of Bridge C 2.1 with time and temperature

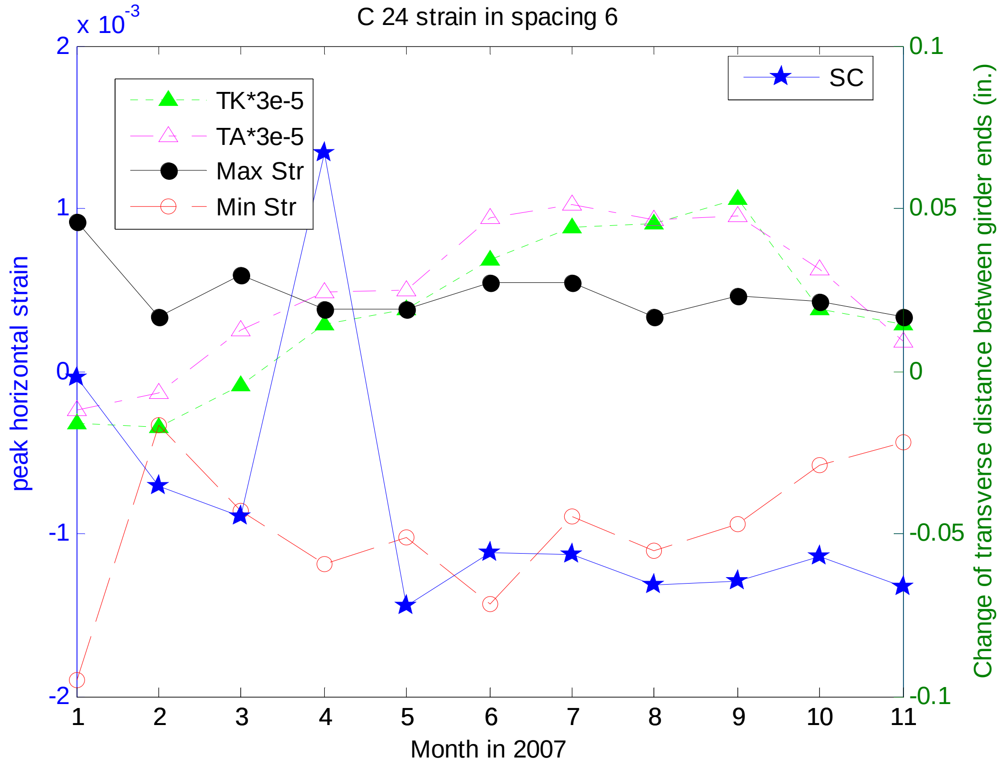


Figure C-134 Variation of peak strain in spacing 6 of Bridge C 2.1 with time and temperature

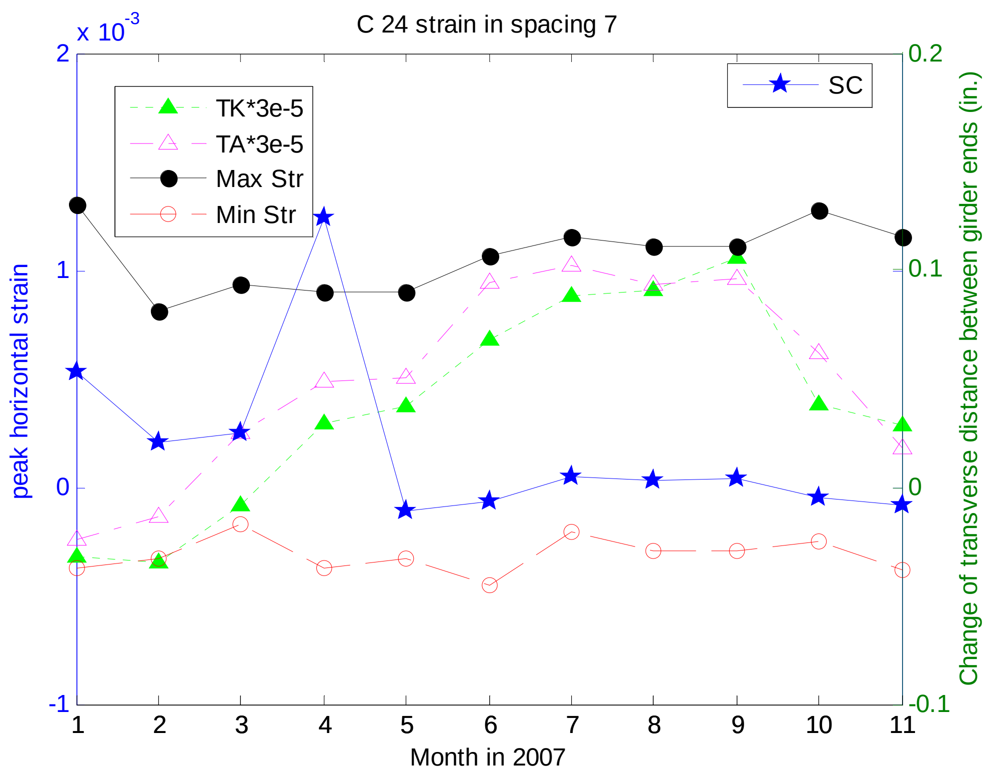


Figure C-135 Variation of peak strain in spacing 7 of Bridge C 2.1 with time and temperature

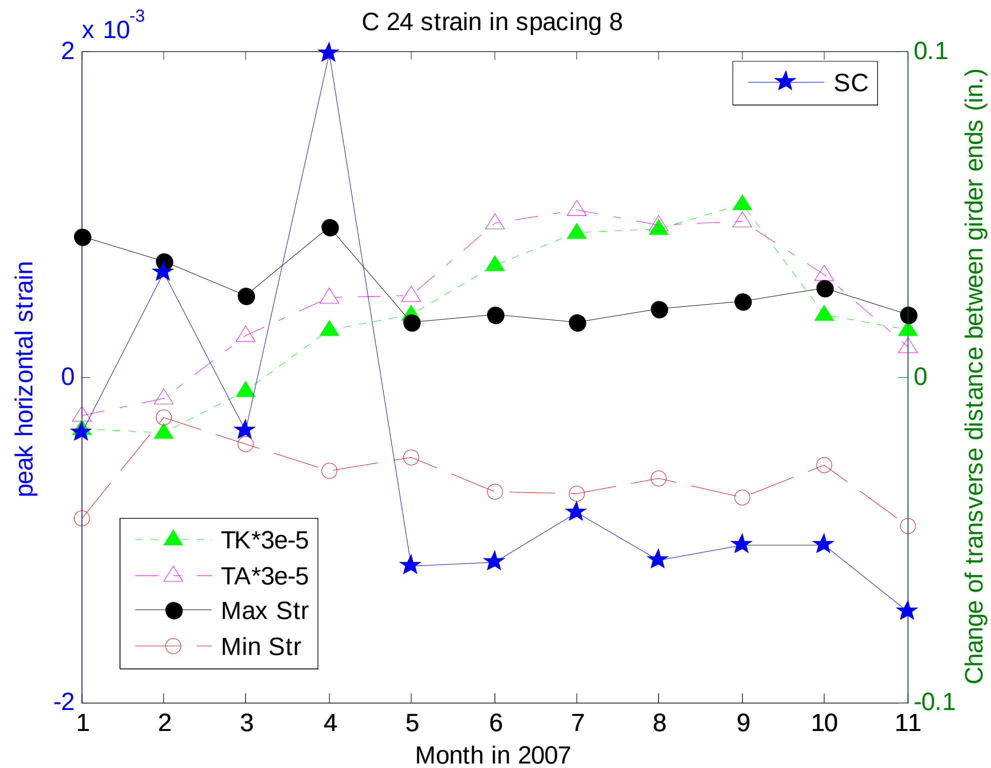


Figure C-136 Variation of peak strain in spacing 8 of Bridge C 2.1 with time and temperature

D. Temperature Fields in FE Simulation

In the finite element simulation of this research (Chapter 5), the temperature fields applied to bridge models were simplified in order to make the simulation of a large number of models feasible. Three simplification approaches and there verification were presented here in this chapter.

D.I Constant Temperature Field in the Deck

D.I.1 Simplification approach

To simplify the application of the temperature gradient to the FE models, the temperature in the deck was assumed to be constant throughout its thickness, so was the temperature in the other parts of the bridge. Thus, the temperature gradient for steel bridges in Figure 5-14 was transformed to the simplified gradient in Figure D-1 by equating the areas under the temperature curves of both figures. The calculation is as follows:

$$\frac{1}{2} \times 30 \times 4 + \frac{1}{2} \times (11 + 6.4) \times 5 + 11 \times 4 = 9 \times T_4$$

$$T_4 = 16.4\ ^{o}\mathrm{F}$$

The temperature gradient in winter is obtained by multiplying the summer gradient by –0.3, which is $T_4^{'} = -0.3 \times T_4 = -4.9\ ^{o}F$. Temperatures at the other parts of the structure are calculated as:

$$\frac{T_6}{11} = \frac{A-5}{A}$$

$$T_6 = T_5 = 6.4 \quad ^{\circ}F$$

In winter time, the temperature in the other part of bridge are taken as $T_6^{'} = T_5^{'} = -0.3 \times T_6 = -1.9\ ^{o}F$.

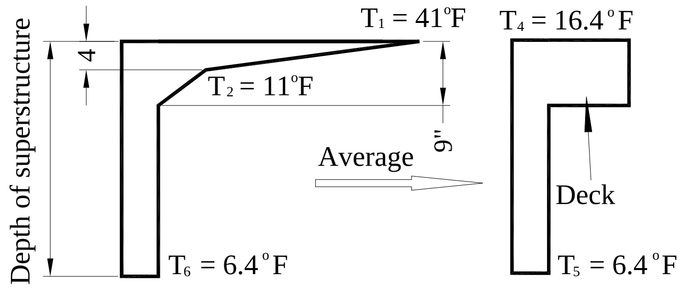


Figure D-1 Simplification of temperature gradient for steel bridges

Similarly, the temperature gradient for concrete bridges in Figure 5-14 was transformed to the simplified gradient shown in Figure D-2. The calculation is as follows:

$$\frac{1}{2}\times 30\times 4+4\times 11+\frac{1}{2}\times 5\times\left(11\times\frac{5}{12}\right)+5\times 11\times\frac{7}{12}=9\times T_4$$

T_4=16.4 ºF

The temperature gradient in winter is the temperature in summer multiplied by –0.3, which is $T_4^{'}$ = -0.3×T_4 = -4.9 *ºF*. The depth of the concrete superstructure is taken to be the average depths of Type I, II, III and IV prestressed I-beams plus the deck thickness:

$$d=\frac{1}{4}\left(28+36+45+54\right)=40.75 \quad (in.)$$

Temperatures at the other parts of the structure are calculated as:

$$\frac{1}{2}\times 7\times\left(11\times\frac{7}{12}\right)=d\times T_5$$

$$T_5=0.6\ ºF$$

The temperature gradient in winter is the temperature in summer multiplied by –0.3, which is $T_5' = -0.3\times T_5 = -0.2\ ºF$.

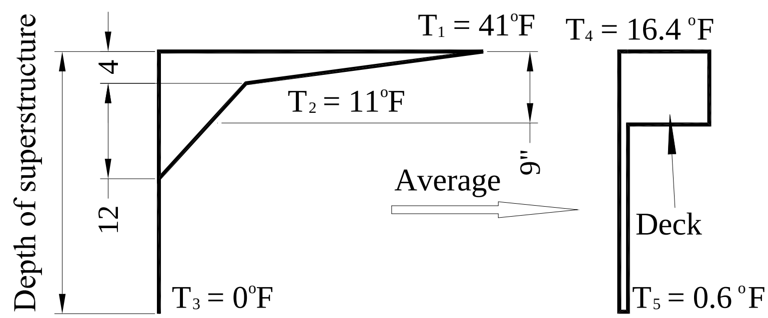


Figure D-2 Simplification of temperature gradient for concrete bridges

D.I.2 Verification of simplified temperature gradient (constant deck temperature)

The effects of the simplified temperature gradient were compared to the LRFD temperature gradient using the example provided by MDOT. The nonlinear thermal stresses induced by simplified temperature gradient on the 48” deep x 24” concrete beam in the examples were calculated and the results are shown in Figure D-3. The temperature value T_3 in the example was changed to be zero, which is the value used in this research.

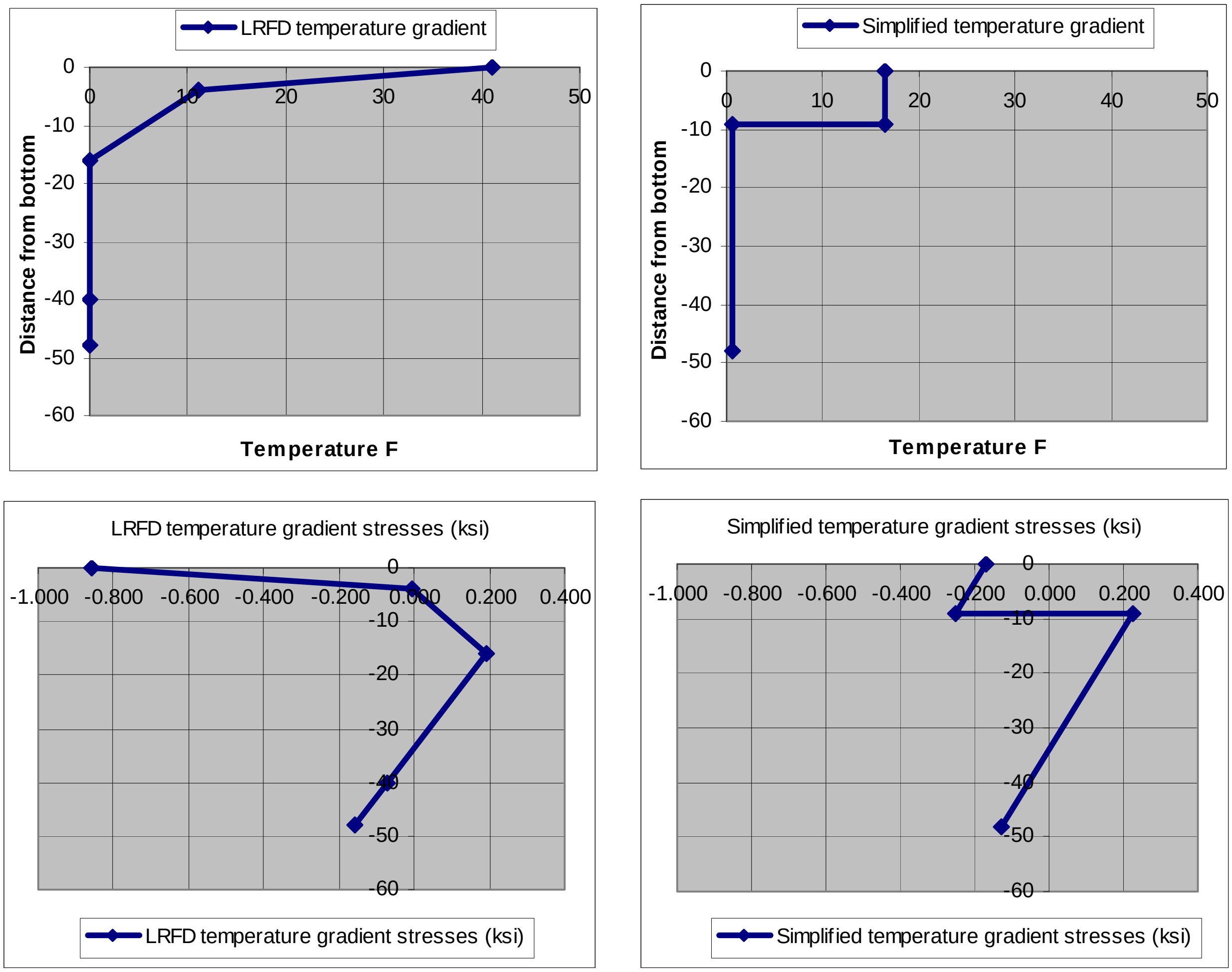


Figure D-3 Comparison stresses induced by actual and simplified temperature gradients

D.II Linear Temperature Gradient in the Deck

D.II.1 Simplification approach

Another approach to a simplified temperature gradient was to assume that the temperature distribution in the deck was linear and the temperature field in other parts of the bridge was constant. Thus, the temperature gradient for steel bridges in Figure 5-14 was transformed to the simplified gradient shown in Figure D-4 Simplification of temperature gradient for steel bridges by equating the areas under the temperature curves of both figures. The calculation is as follows:

$$\left[\frac{1}{2}\times 30\times 4\right]+\left[\frac{1}{2}\times(11-6.4)\times 5\right]+[(11-6.4)\times 4]=\frac{1}{2}\times 9\times(T_4-6.4)$$

$$T_4 = 26.4\ ^{o}F$$

The temperature gradient in winter is obtained by multiplying the summer gradient by –0.3, which is $T_4^{'} = -0.3\times T_4 = -7.9\ ^{o}F$. Temperatures at the other parts of the structure are calculated as:

$$\frac{T_6}{11}=\frac{A-5}{A}$$

$$T_6 = T_5 = 6.4 \quad ^{\circ}F$$

In winter time, the temperature in the other parts of the bridge is taken as:

$$T_6^{'} = T_5^{'} = -0.3\times T_6 = -1.9\ ^{o}F$$

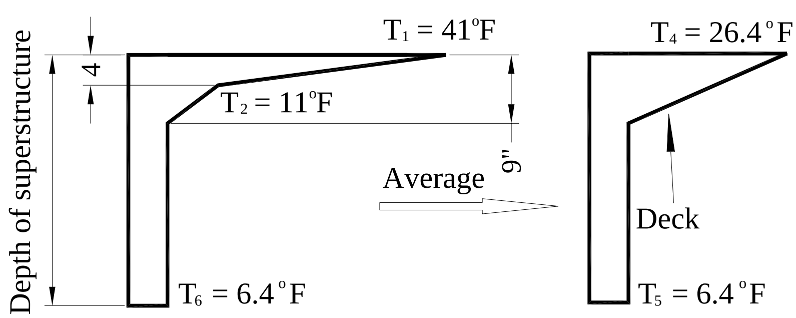


Figure D-4 Simplification of temperature gradient for steel bridges

Similarly, the temperature gradient for concrete bridges in Figure 5-14 was transformed to the simplified gradient shown in Figure D-2. The calculation is as follows:

$$\left[\frac{1}{2}\times 30\times 4\right]+[4\times 11]+\left[\frac{1}{2}\times 5\times\left(11\times\frac{5}{12}\right)\right]+\left[5\times 11\times\frac{7}{12}\right]=\left[\frac{1}{2}\times 9\times(T_4-0.6)\right]+[0.6\times 9]$$

$$T_4 = 32.2\ ^{o}\mathrm{F}$$

The temperature gradient in winter is the temperature in summer multiplied by –0.3, which is $T_4^{'}$ = -0.3×T_4 = -9.7 *ºF*. The depth of the concrete superstructure is taken to be the average depths of Type I, II, III and IV prestressed I beams plus the deck thickness:

$$d=\frac{1}{4}(28+36+45+54)=40.75\quad (in.)$$

Temperatures at other parts of the structure are calculated as:

$$\frac{1}{2}\times 7\times\left(11\times\frac{7}{12}\right)=d\times T_5$$

$$T_5=0.6\ ^{o}\mathrm{F}$$

The temperature gradient in winter is the temperature in summer multiplied by –0.3, which is:

$$T_5^{'} = -0.3\times T_5 = -0.2\ ^{o}F.$$

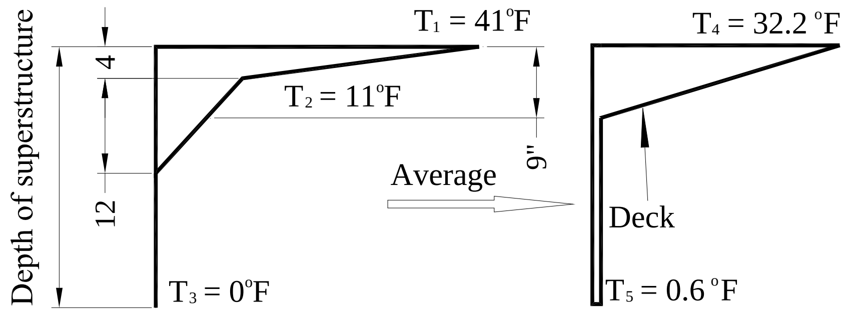


Figure D-5 Simplification of temperature gradient for concrete bridges

D.II.2 Verification of simplified temperature gradient (linear deck temperature)

The effects of the simplified temperature gradient are compared to the LRFD temperature gradient using the example provided by MDOT. The nonlinear thermal stresses induced by the simplified temperature gradient on the 48” deep x 24” concrete beam in the examples were calculated and the results are shown in Figure D-6. The temperature value T_3 in the example is changed to be zero, which is the value used in this research.

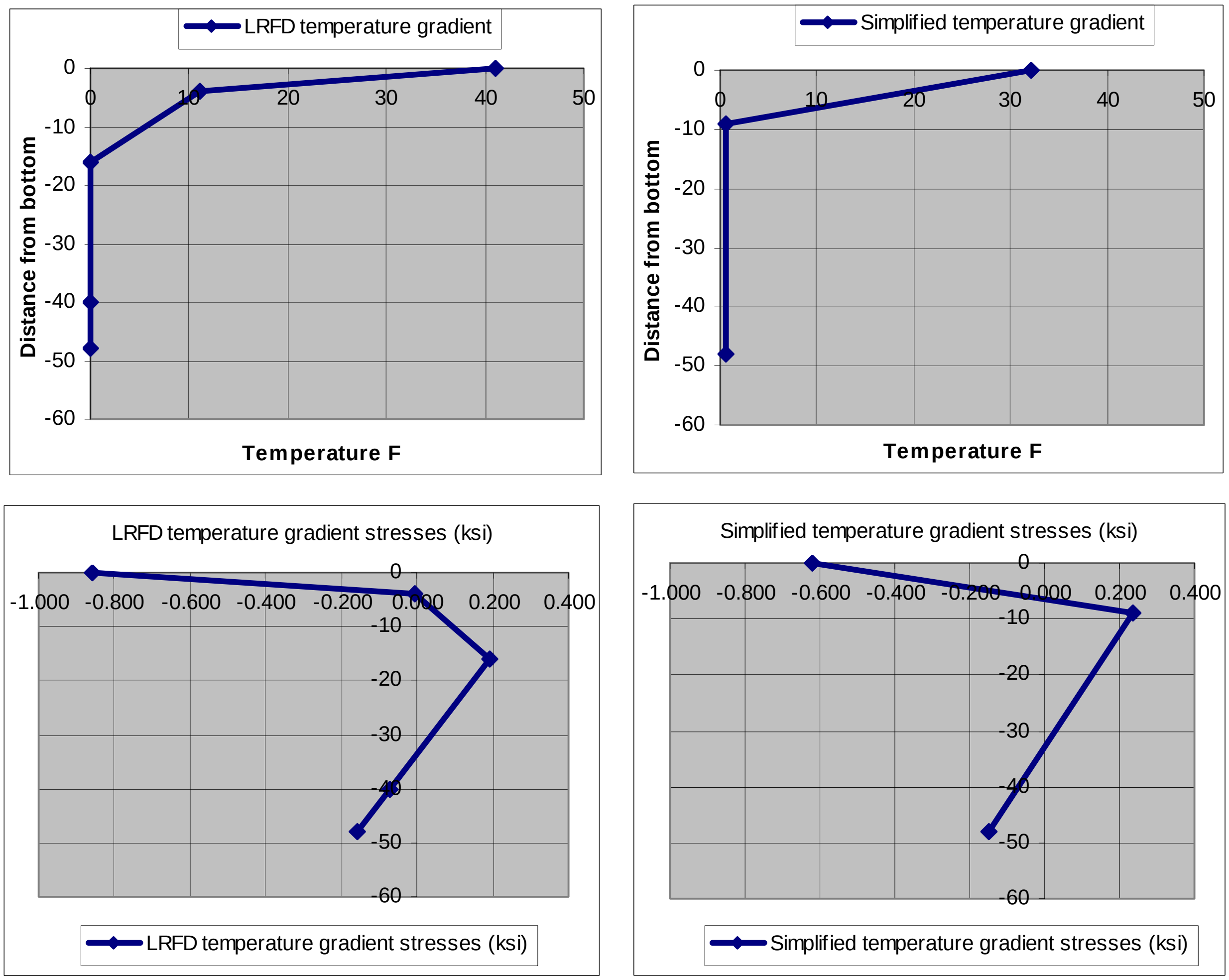


Figure D-6 Comparison stresses induced by the actual and simplified temperature gradients

D.III Linear Temperature Field with Temperature at the Top of the Deck unchanged

D.III.1 Simplification approach

The third temperature field proposed is also a linear temperature field throughout deck thickness. The temperature at the top of the deck is taken to be the value of the LRFD value. The temperature at the bottom of the deck is taken to be same as the simplified value of the rest of the structure (the temperature in concrete girder is taken to be zero for this simplification approach.) The simplified temperature fields for steel and concrete structure are shown in Figure D-7 and Figure D-8 respectively.

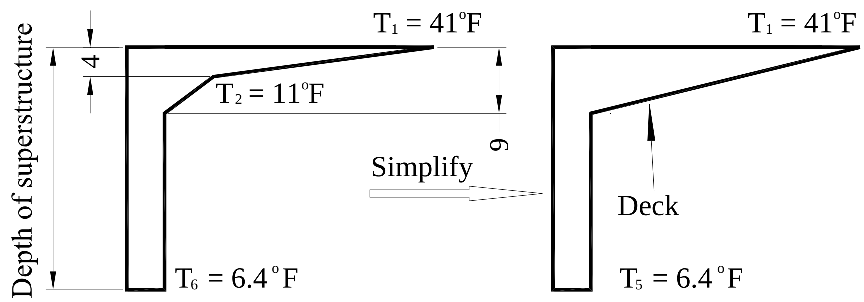


Figure D-7 Simplification of temperature gradient for steel bridges

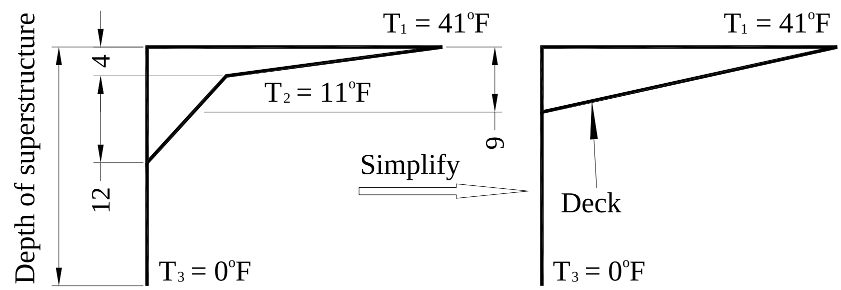


Figure D-8 Simplification of temperature gradient for concrete bridges

D.III.2 Verification of simplified temperature gradient (constant deck temperature)

The effects of the simplified temperature gradient were compared to the LRFD temperature gradient using the example provided by MDOT. The nonlinear thermal stresses induced by simplified temperature gradient on the 48” deep x 24” concrete beam in the examples were calculated and the results are shown in Figure D-9. The temperature value T_3 in the example was changed to be zero, which is the value used in this research.

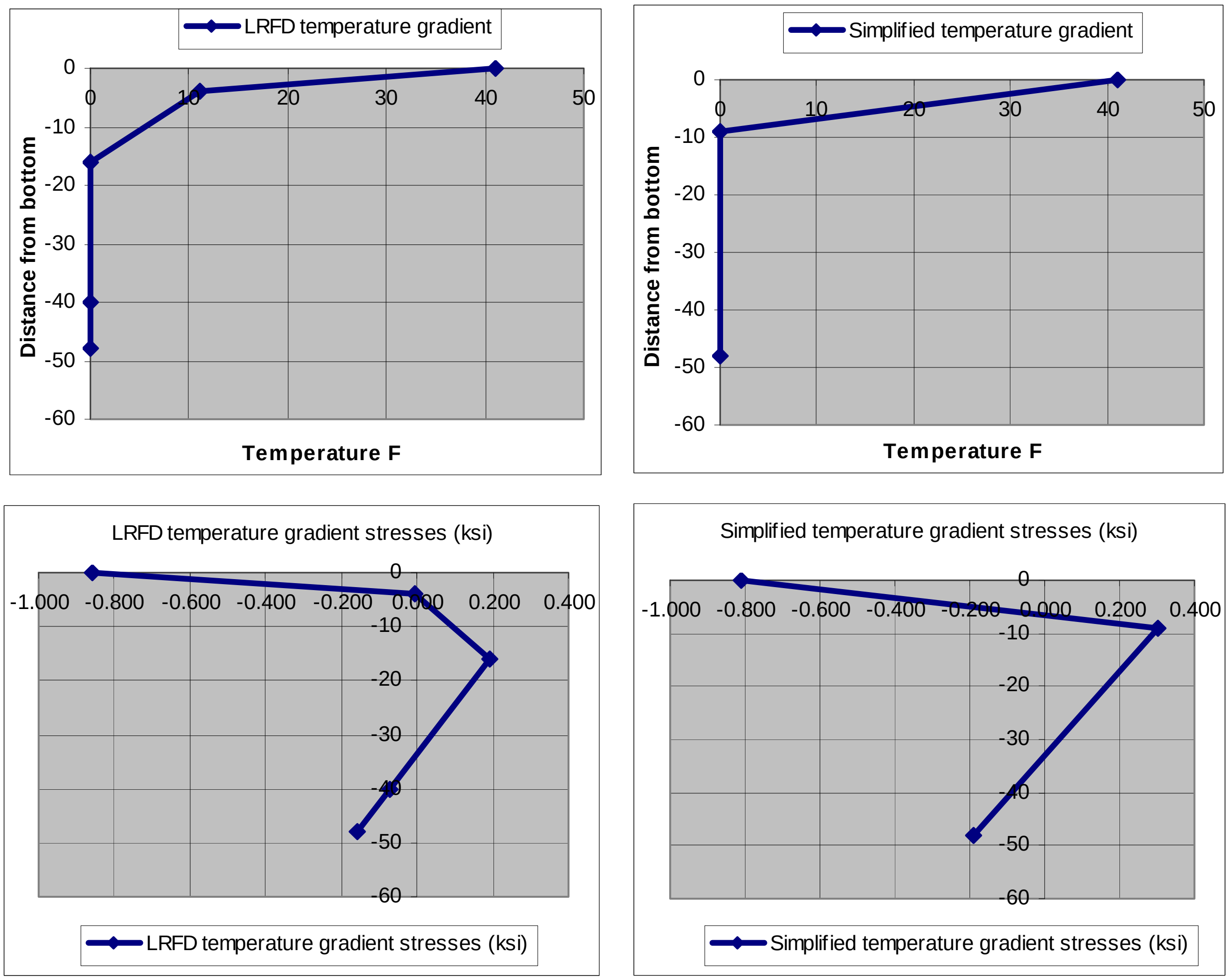


Figure D-9 Comparison stresses induced by the actual and simplified temperature gradients

D.IV Verification Results Summary

The comparison of temperature-induced axial strains and curvatures is shown in Table D-1. It can be determined from Figure D-6 and Table D-1 that the simplified linear temperature gradient is a reasonable approximation of the LRFD temperature gradient.

Table D-1 Comparison axial strains and curvatures induced by temperature gradient

Temperature field	Axial strain	Strain error ratio	Curvature	Curvature error ratio
LRFD	2.13E-05		-2.20E-06	
Constant in the deck	2.13E-05	0.2%	-1.81E-06	17.6%
Linear gradient in the deck	2.14E-05	0.6%	-1.99E-06	9.5%
Linear gradient in the deck (with original top value)	2.31E-05	8.5%	-2.52E-06	14.5%

D.V Temperature Values in the Simulation

It can be determined from Table D-1 that the approximation is better when using linear temperature gradient in the deck and constant temperature field in the other part of the structure based on the equivalence of the areas under the temperature curves. The values for the simulations are summarized in Table D-2.

Table D-2 Temperature values for linear temperature gradient in the deck

Structures (Members)		Construction (Fahrenheit degree)	Winter (Fahrenheit degree)	Summer (Fahrenheit degree)
Steel Bridge	Top of the deck	60.0	-37.9	146.4
	Other members	60.0	-31.9	126.4
Concrete Bridge	Top of the deck	60.0	-9.7	112.2
	Other members	60.0	-0.2	80.6

E. Finite Element Simulation Results

A complete set of finite element simulation (Chapter 5) results was presented in this chapter. Bridges of each main structural type were organized in a separate section.

E.I Simple or Cantilevered Steel Bridges

The largest maximum principal stresses in the specified region of abutment walls of simple/cantilevered steel bridges were shown in this section, Figure E-1 to **Figure** E-6 show bridges with deck width of 42.5 ft, **Figure** E-7 to **Figure** E-12 and **Figure** E-13 to **Figure** E-18 show bridges with deck width of 58.5 ft and 74.5 ft; respectively. The largest horizontal strains in the specified region of abutment walls of simple/cantilevered steel bridges were also plotted in this section, **Figure** E-19 to **Figure** E-24 show bridges with deck width of 42.5 ft, **Figure** E-25 to **Figure** E-30 and **Figure** E-31 to **Figure** E-36 show bridges with deck width of 58.5 ft and 74.5 ft; respectively.

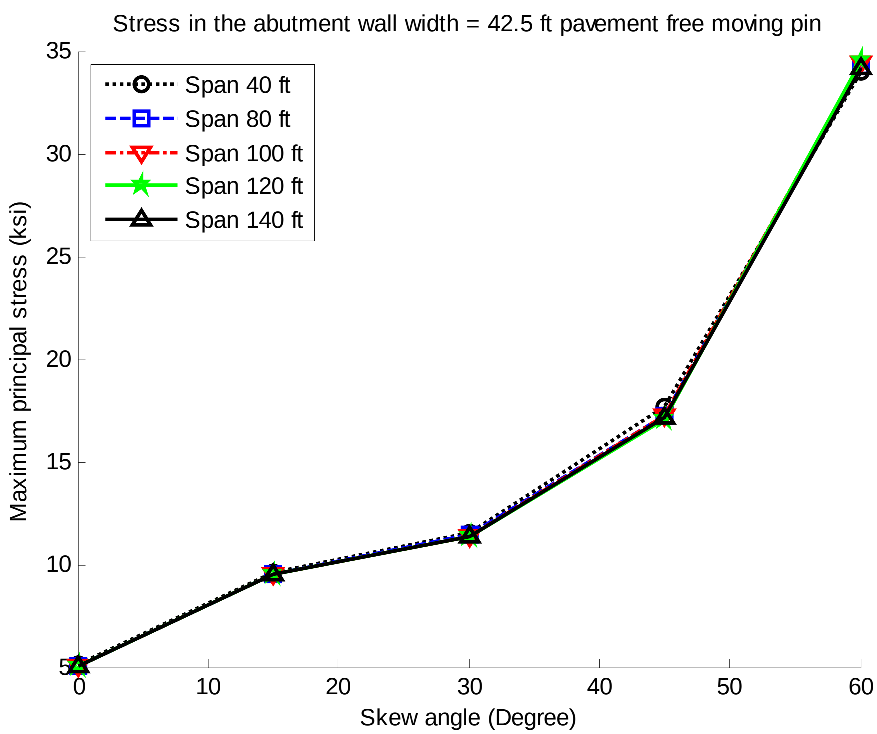


Figure E-1 Maximum stress of bridges under pavement pressure (width = 42.5 ft, free moving pin and hanger)

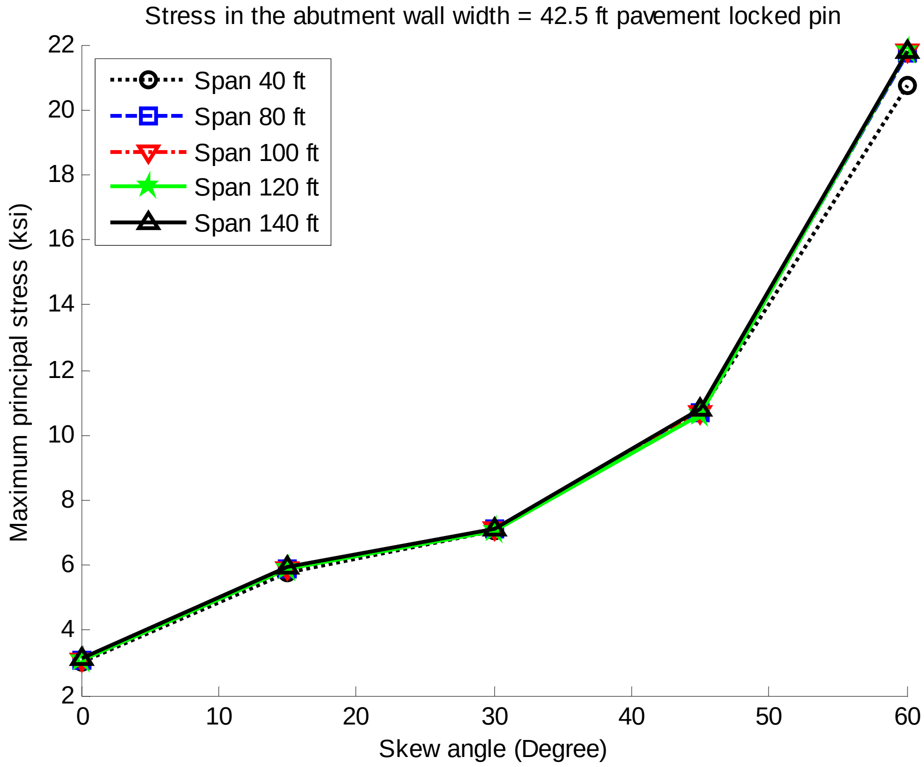


Figure E-2 Maximum stress of bridges under pavement pressure (width = 42.5 ft, pin and hanger locked)

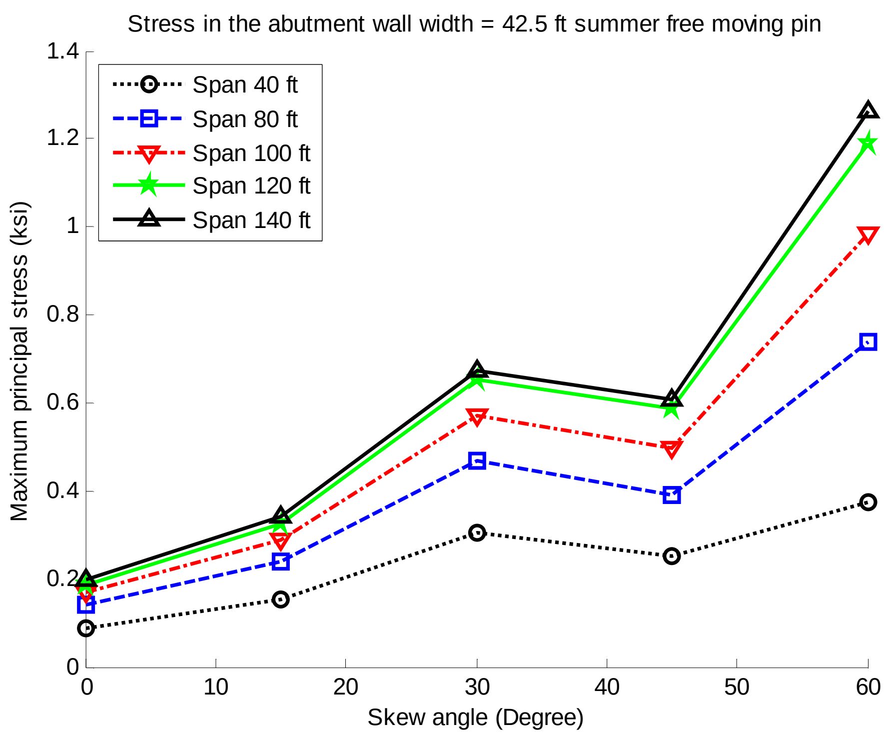


Figure E-3 Maximum stress of bridges under summer temperature (width = 42.5 ft, free moving pin and hanger)

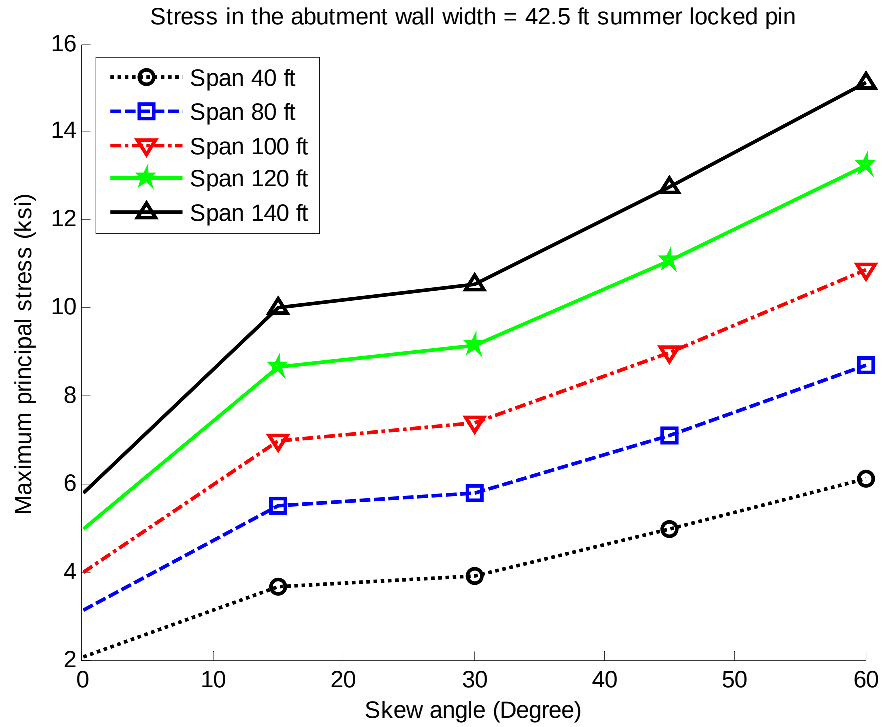


Figure E-4 Maximum stress of bridges under summer temperature (width = 42.5 ft, pin and hanger locked)

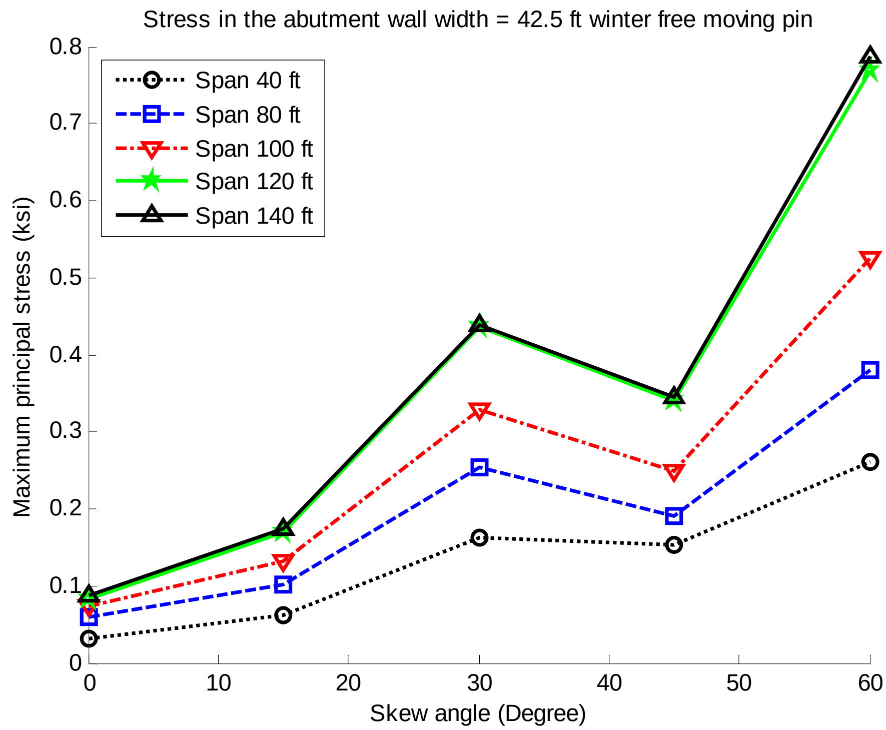


Figure E-5 Maximum stress of bridges under winter temperature (width = 42.5 ft, free moving pin and hanger)

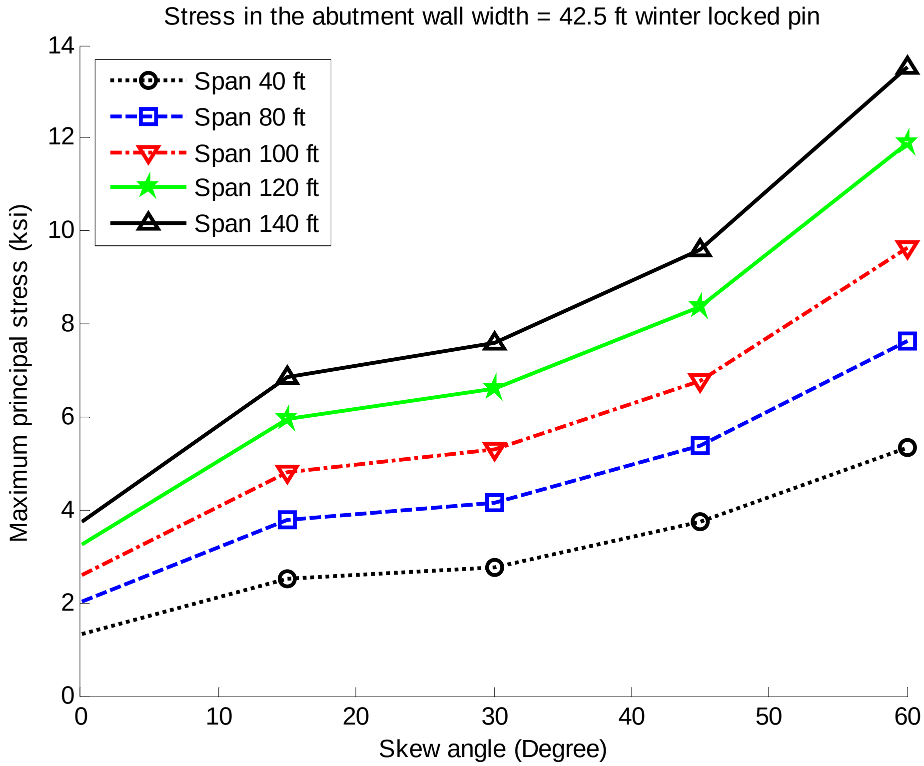


Figure E-6 Maximum stress of bridges under winter temperature (width = 42.5 ft, pin and hanger locked)

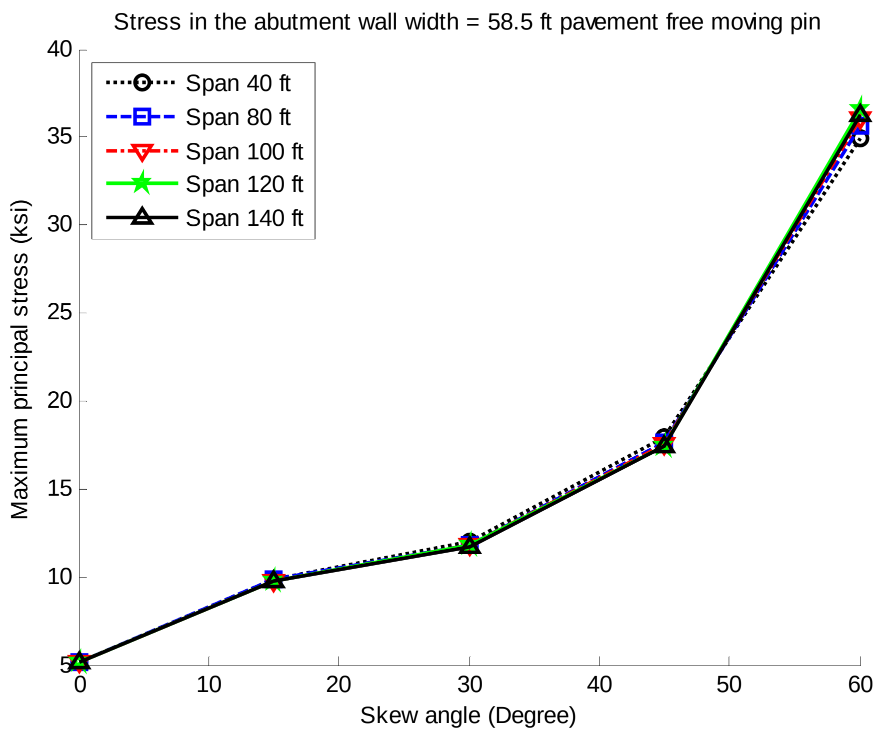


Figure E-7 Maximum stress of bridges under pavement pressure (width = 58.5 ft, free moving pin and hanger)

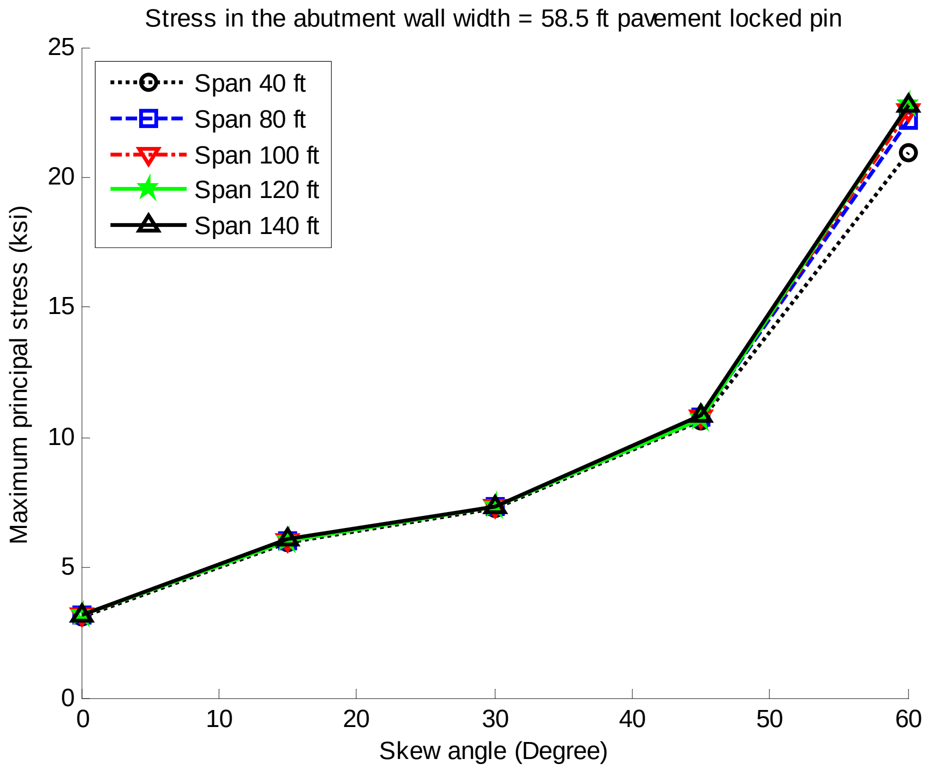


Figure E-8 Maximum stress of bridges under pavement pressure (width = 58.5 ft, pin and hanger locked)

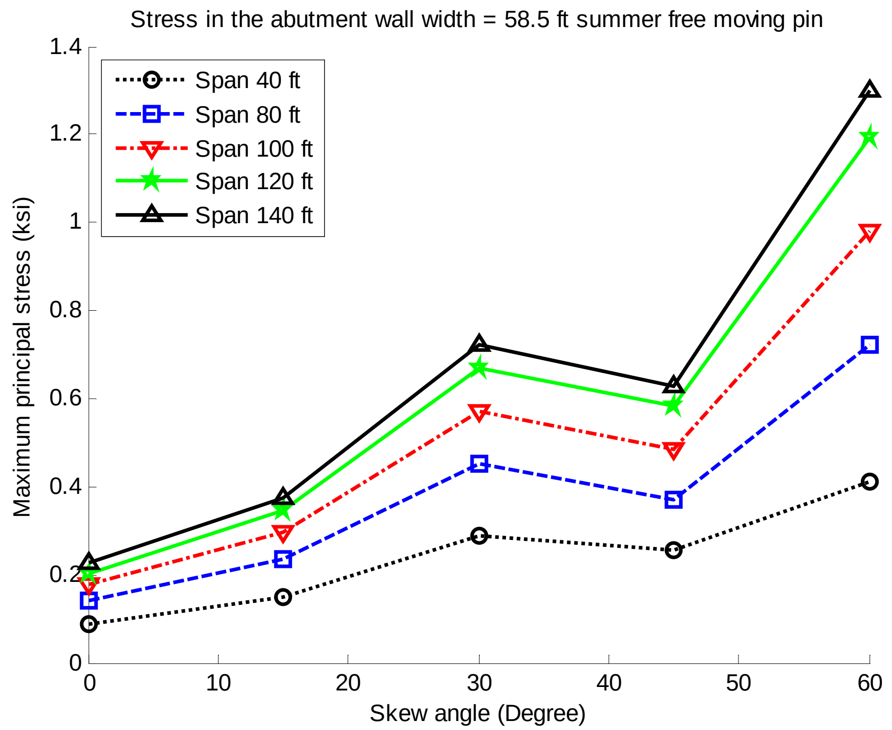


Figure E-9 Maximum stress of bridges under summer temperature (width = 58.5 ft, free moving pin and hanger)

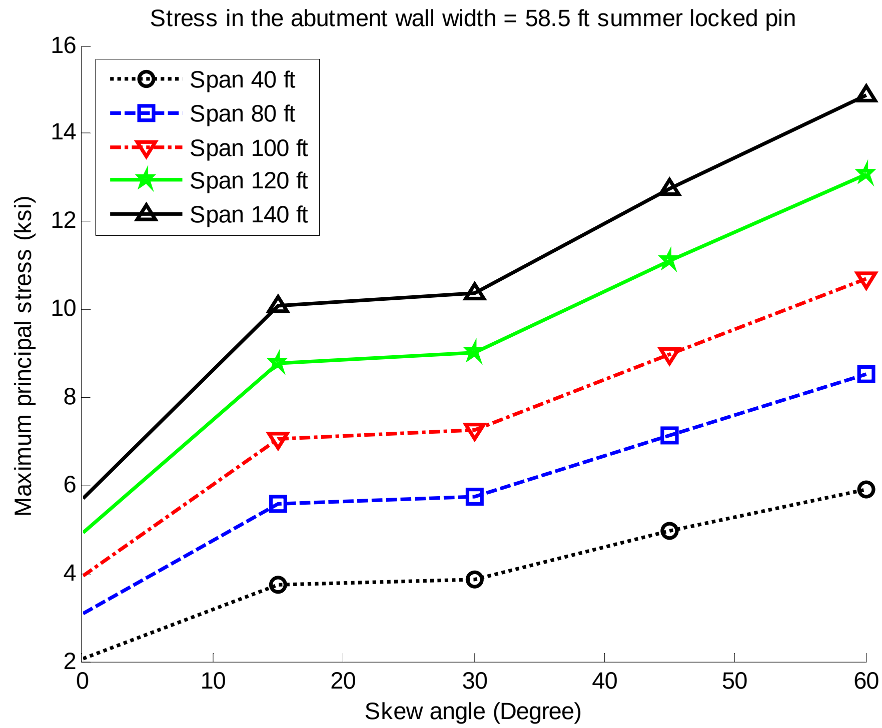


Figure E-10 Maximum stress of bridges under summer temperature (width = 58.5 ft, pin and hanger locked)

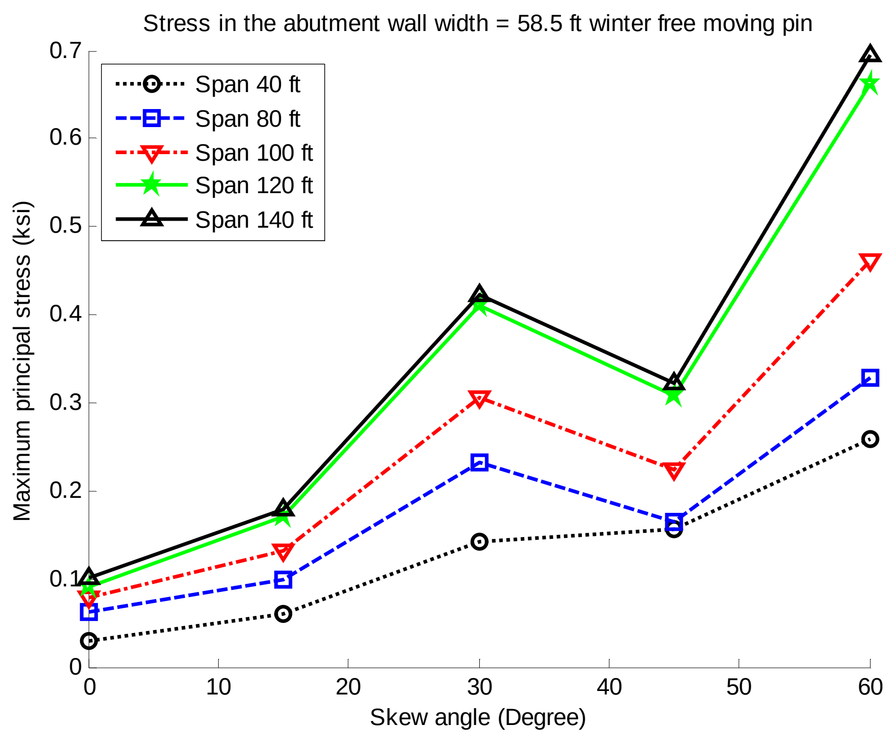


Figure E-11 Maximum stress of bridges under winter temperature (width = 58.5 ft, free moving pin and hanger)

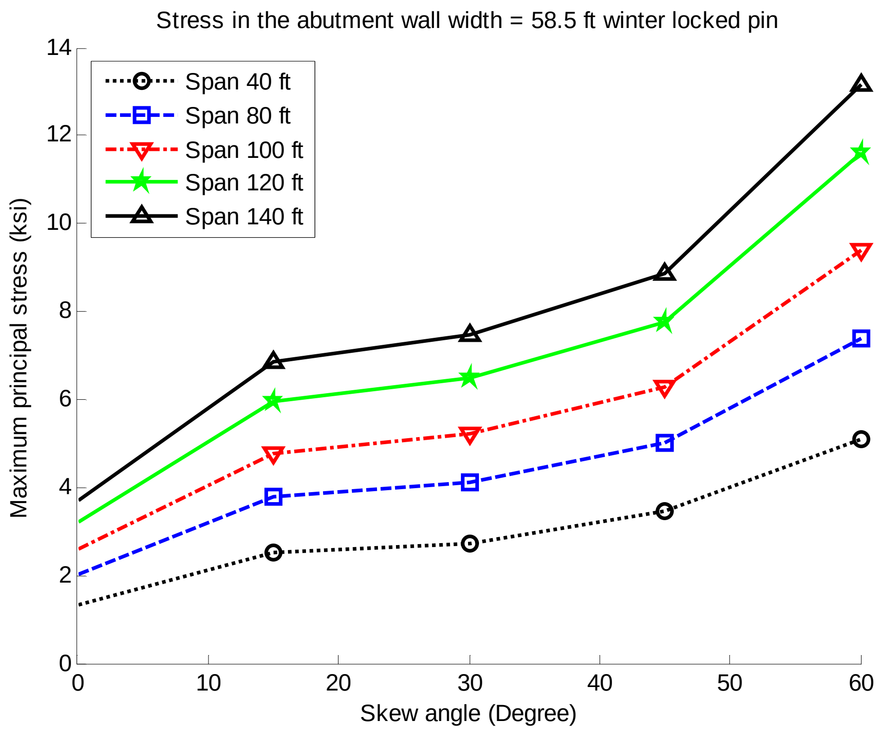


Figure E-12 Maximum stress of bridges under winter temperature (width = 58.5 ft, pin and hanger locked)

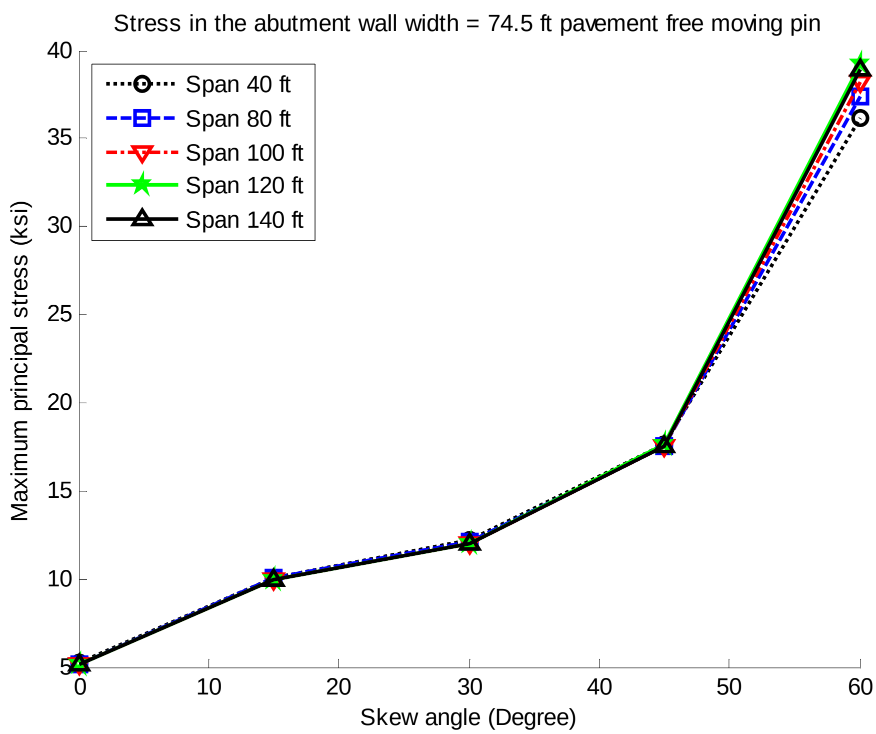


Figure E-13 Maximum stress of bridges under pavement pressure (width = 74.5 ft, free moving pin and hanger)

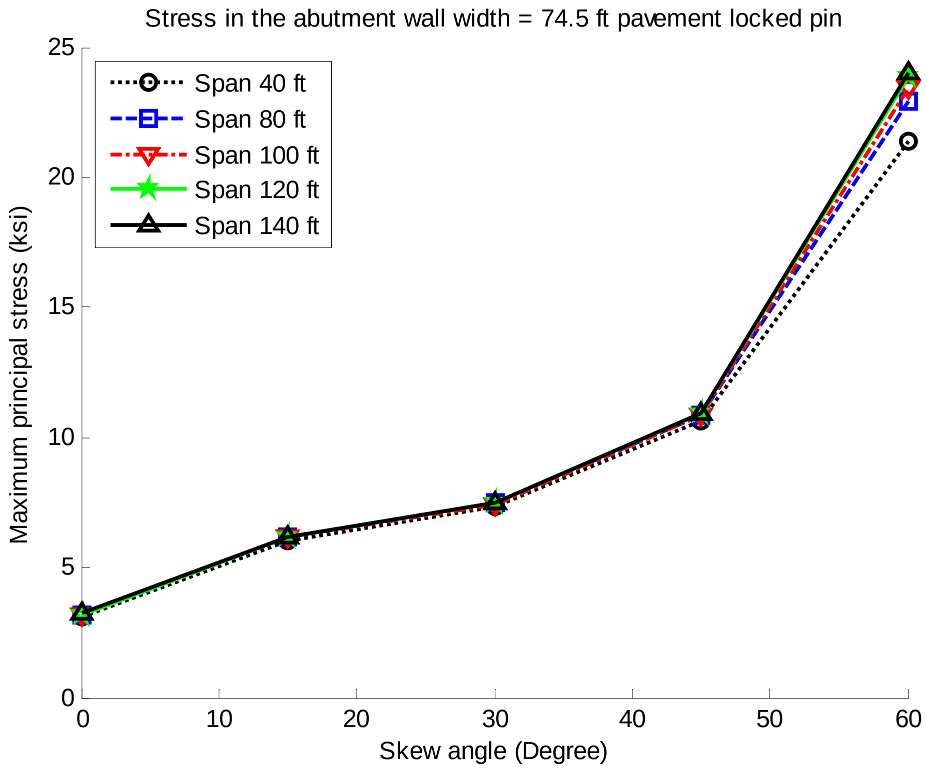


Figure E-14 Maximum stress of bridges under pavement pressure (width = 74.5 ft, pin and hanger locked)

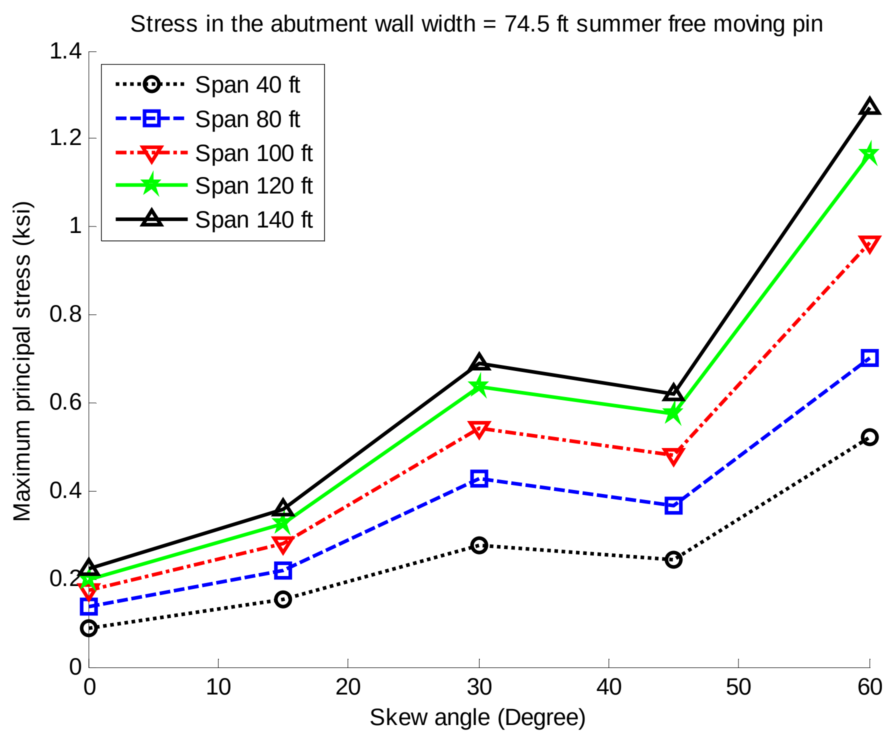


Figure E-15 Maximum stress of bridges under summer temperature (width = 74.5 ft, free moving pin and hanger)

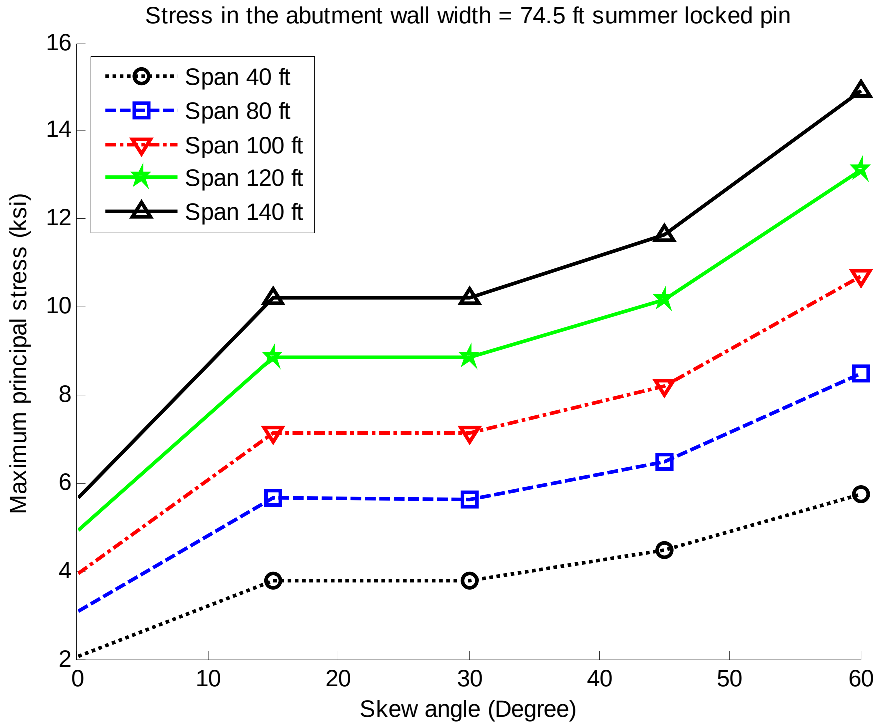


Figure E-16 Maximum stress of bridges under summer temperature (width = 74.5 ft, pin and hanger locked)

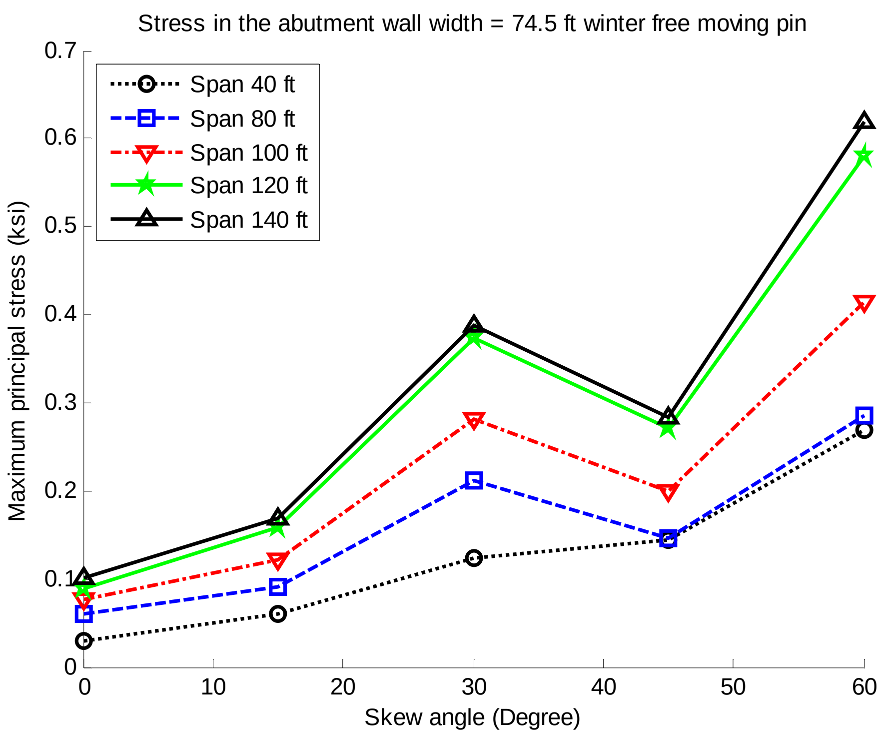


Figure E-17 Maximum stress of bridges under winter temperature (width = 74.5 ft, free moving pin and hanger)

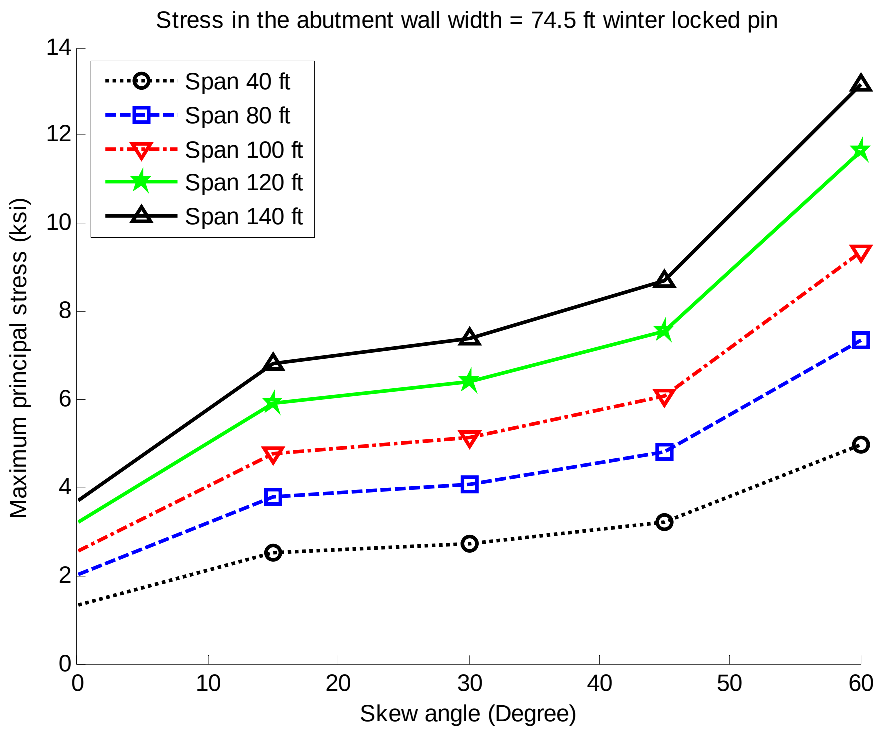


Figure E-18 Maximum stress of bridges under winter temperature (width = 74.5 ft, pin and hanger locked)

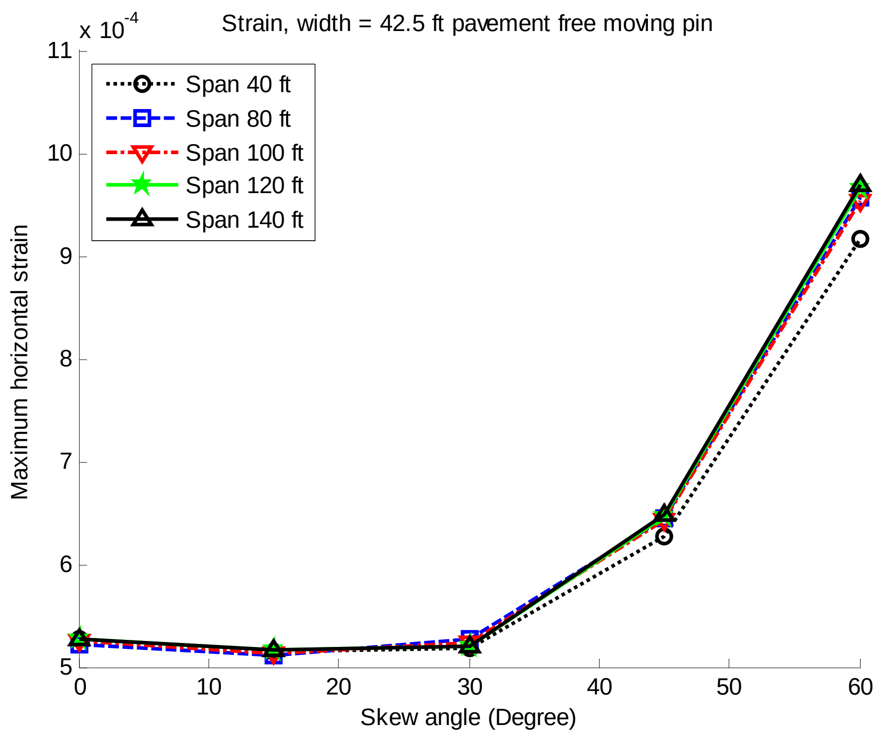


Figure E-19 Maximum strain of bridges under pavement pressure (width = 42.5 ft, free moving pin and hanger)

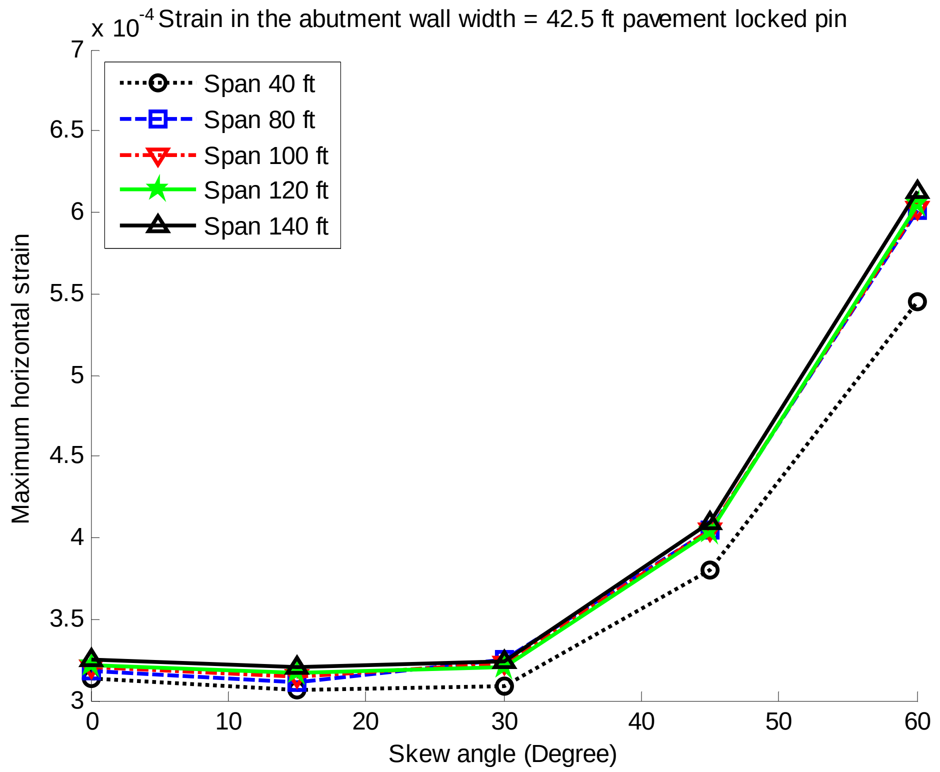


Figure E-20 Maximum strain of bridges under pavement pressure (width = 42.5 ft, pin and hanger locked)

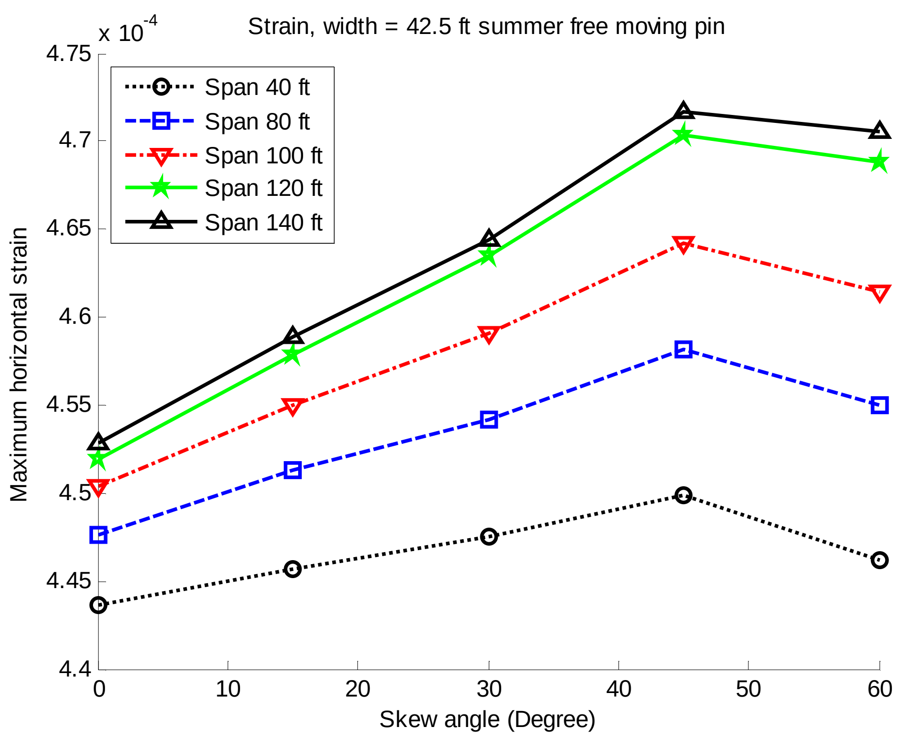


Figure E-21 Maximum strain of bridges under summer temperature (width = 42.5 ft, free moving pin and hanger)

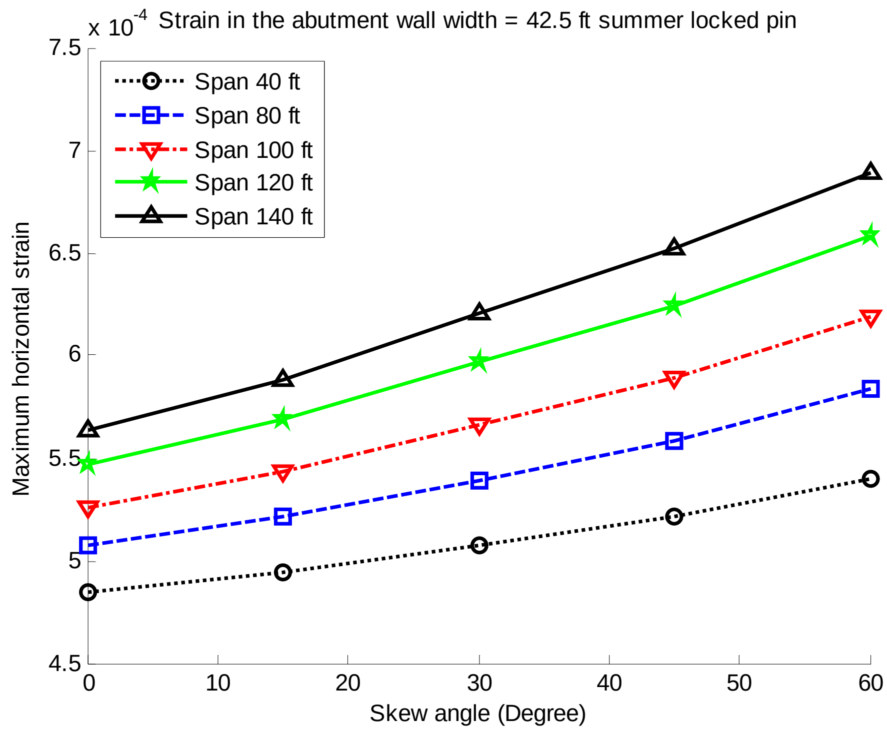


Figure E-22 Maximum strain of bridges under summer temperature (width = 42.5 ft, pin and hanger locked)

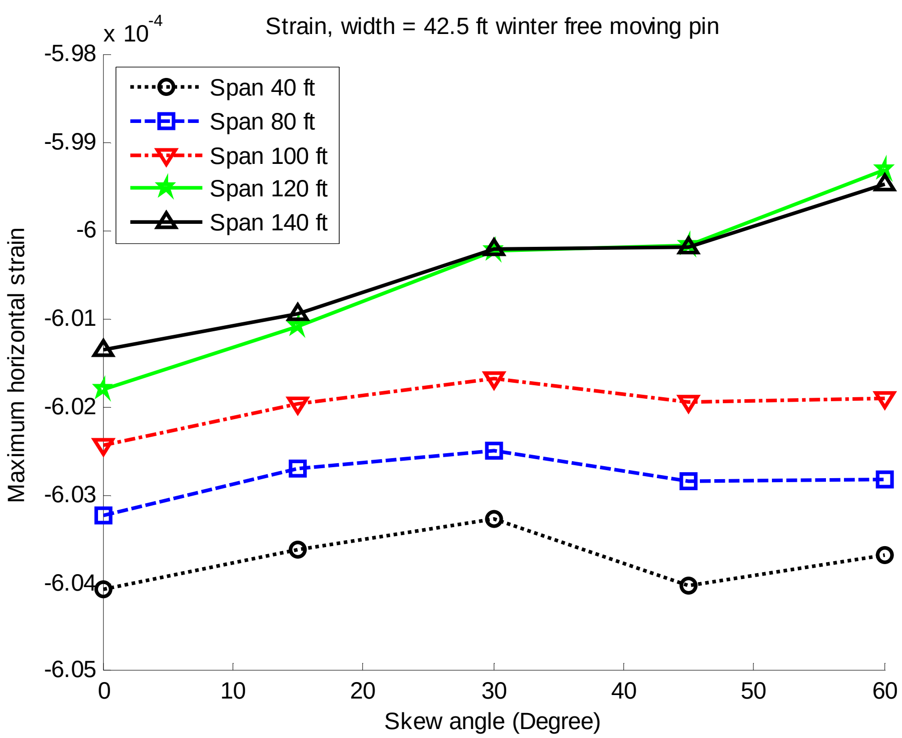


Figure E-23 Maximum strain of bridges under winter temperature (width = 42.5 ft, free moving pin and hanger)

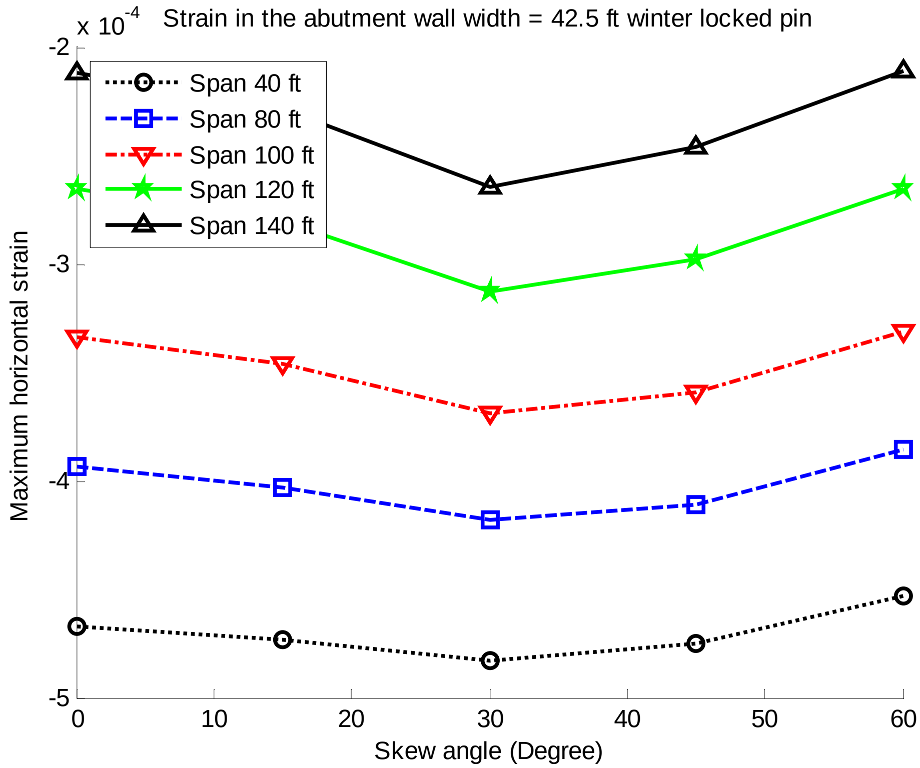


Figure E-24 Maximum strain of bridges under winter temperature (width = 42.5 ft, pin and hanger locked)

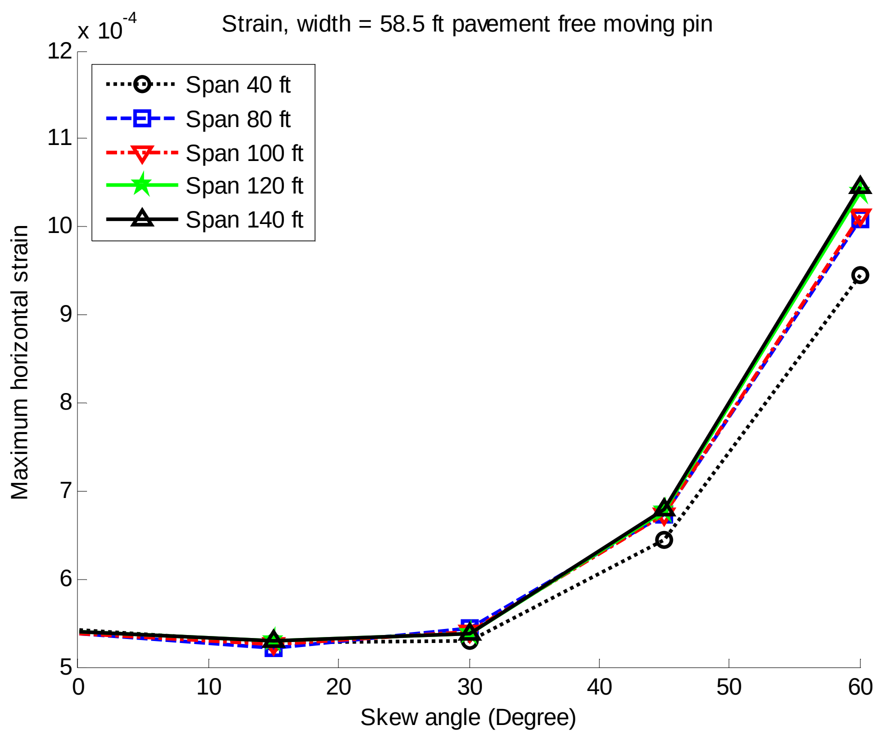


Figure E-25 Maximum strain of bridges under pavement pressure (width = 58.5 ft, free moving pin and hanger)

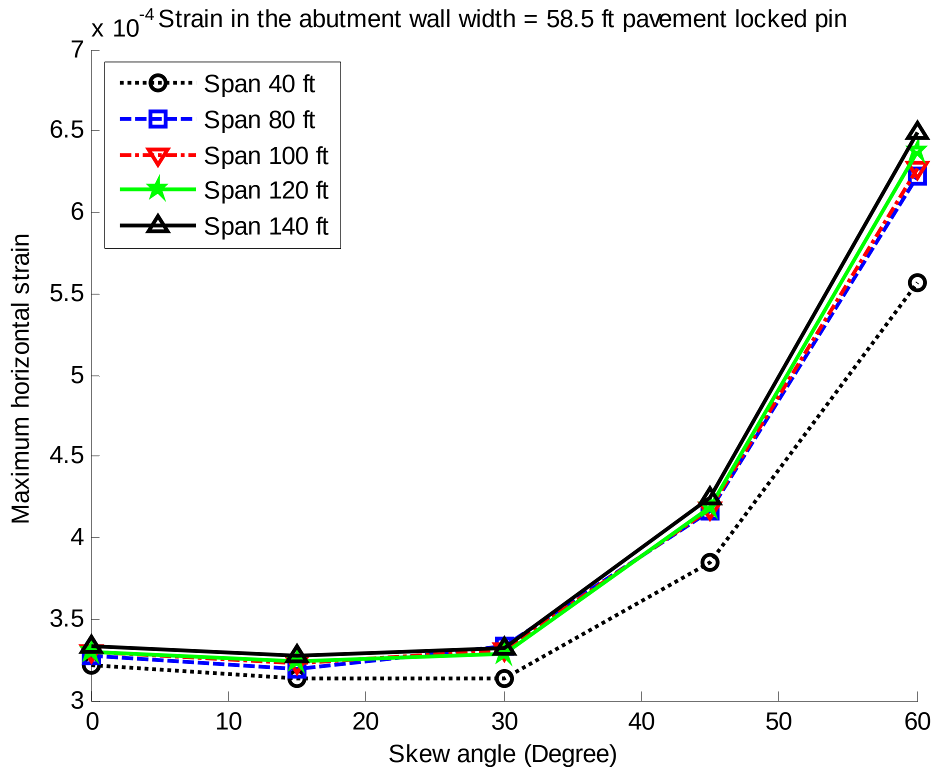


Figure E-26 Maximum strain of bridges under pavement pressure (width = 58.5 ft, pin and hanger locked)

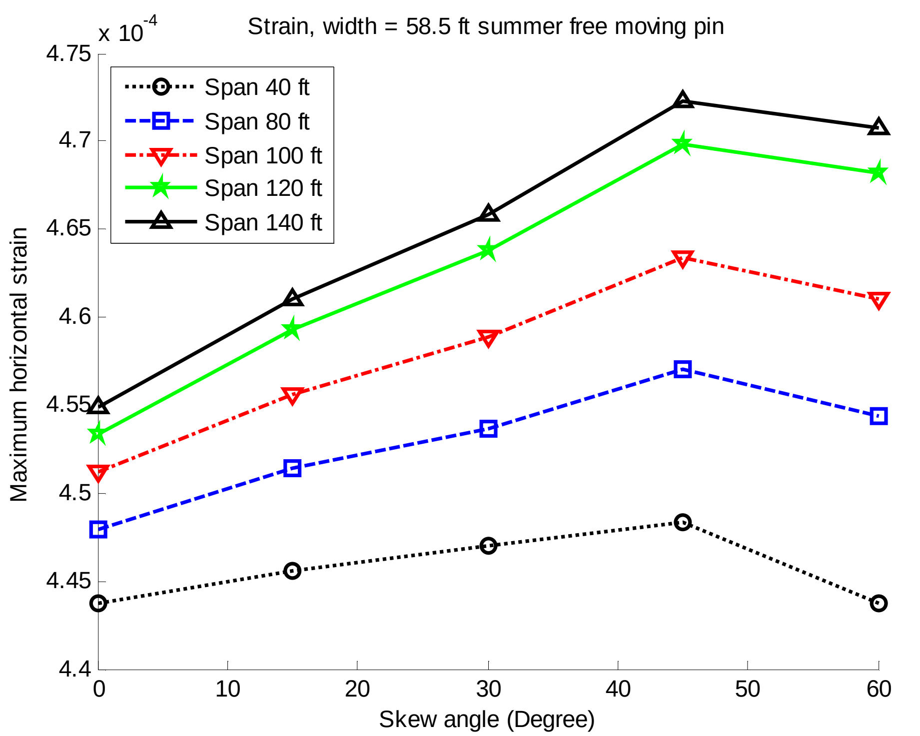


Figure E-27 Maximum strain of bridges under summer temperature (width = 58.5 ft, free moving pin and hanger)

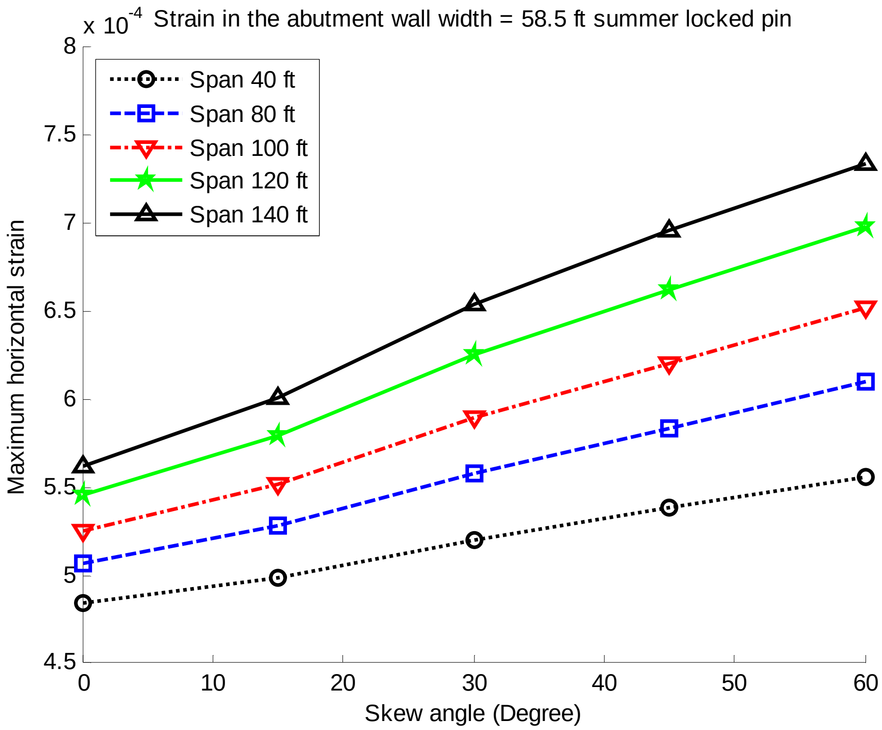


Figure E-28 Maximum strain of bridges under summer temperature (width = 58.5 ft, pin and hanger locked)

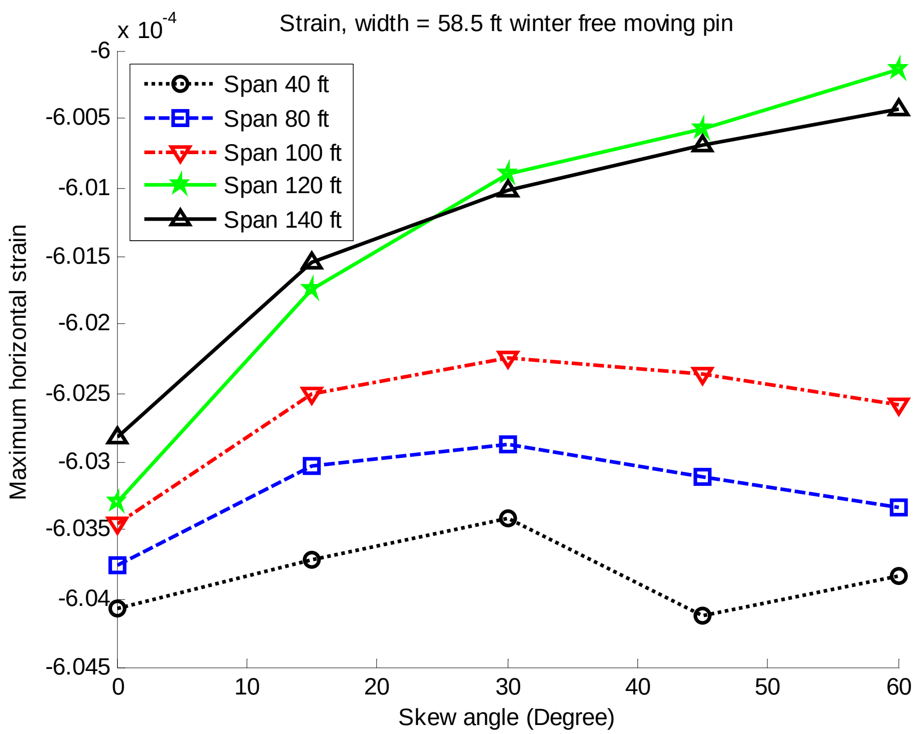


Figure E-29 Maximum strain of bridges under winter temperature (width = 58.5 ft, free moving pin and hanger)

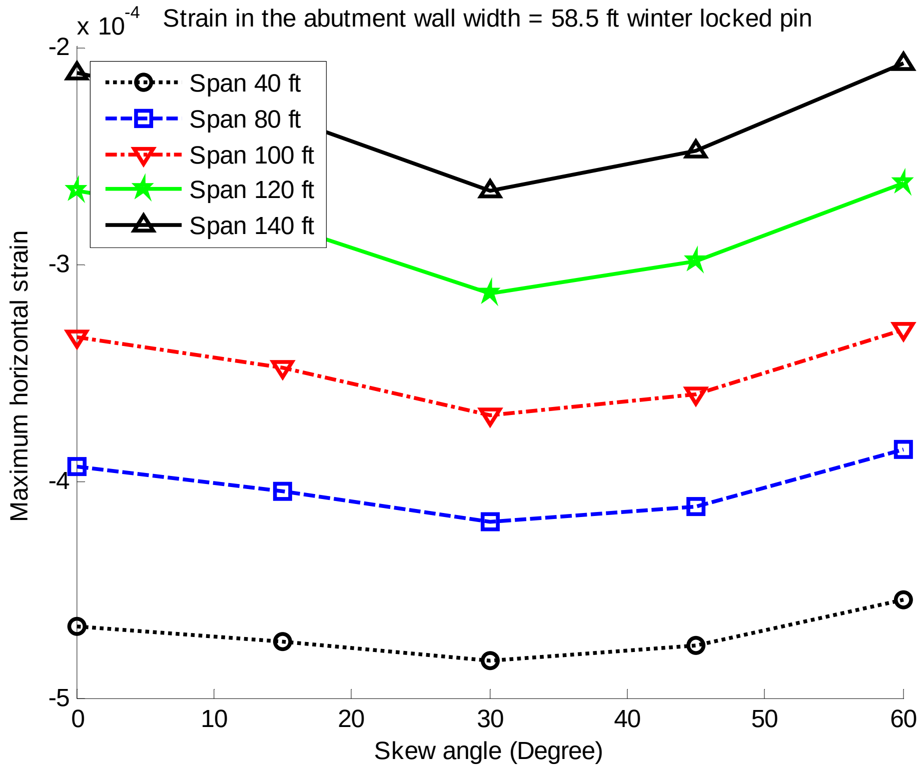


Figure E-30 Maximum strain of bridges under winter temperature (width = 58.5 ft, pin and hanger locked)

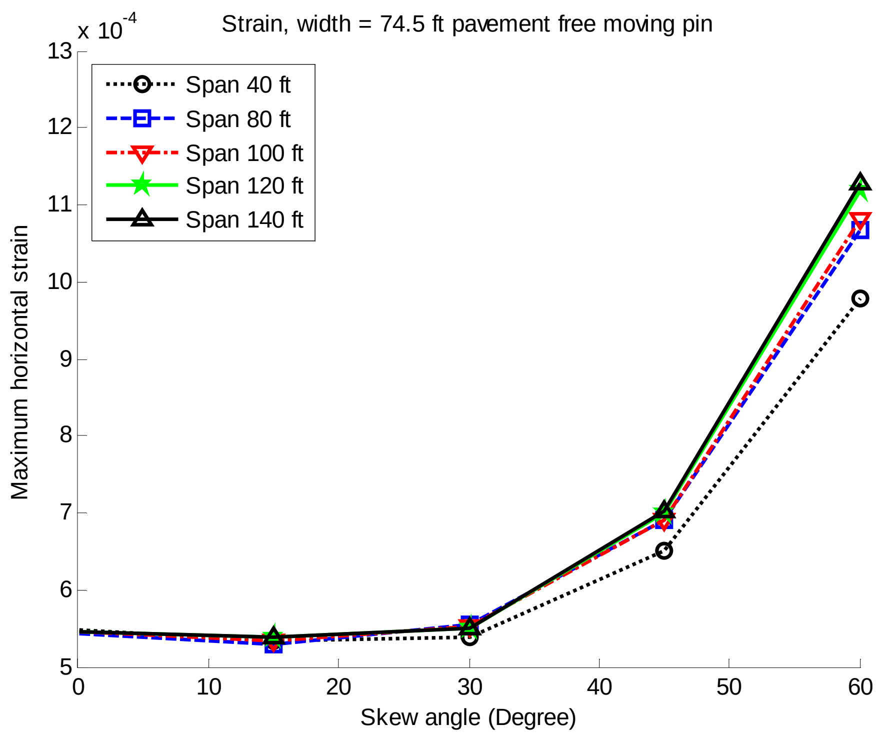


Figure E-31 Maximum strain of bridges under pavement pressure (width = 74.5 ft, free moving pin and hanger)

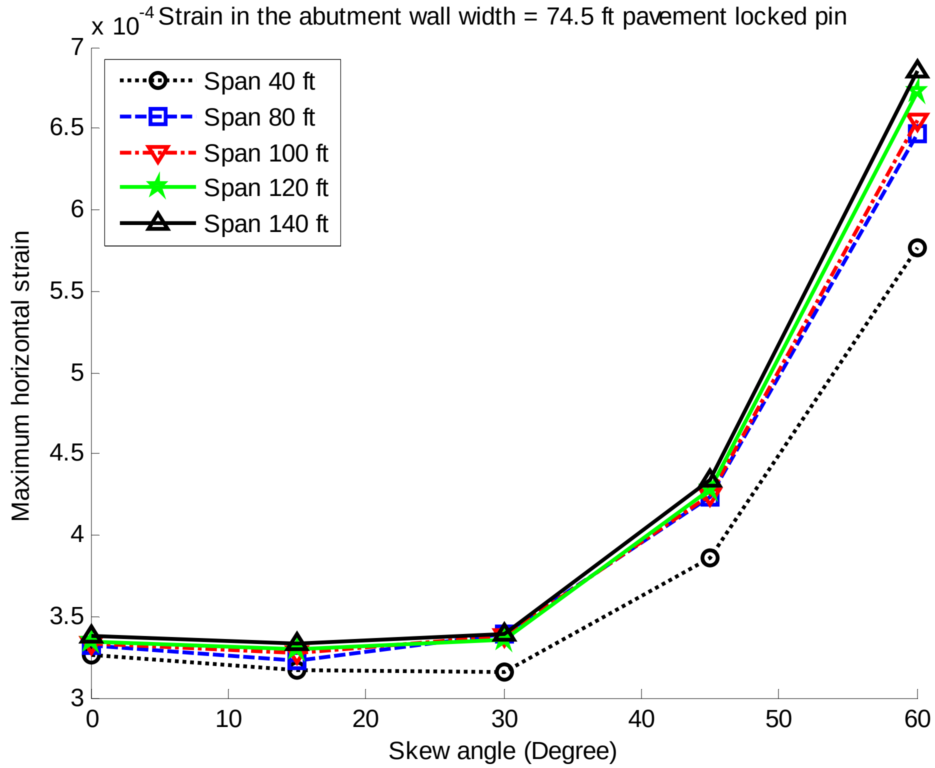


Figure E-32 Maximum strain of bridges under pavement pressure (width = 74.5 ft, pin and hanger locked)

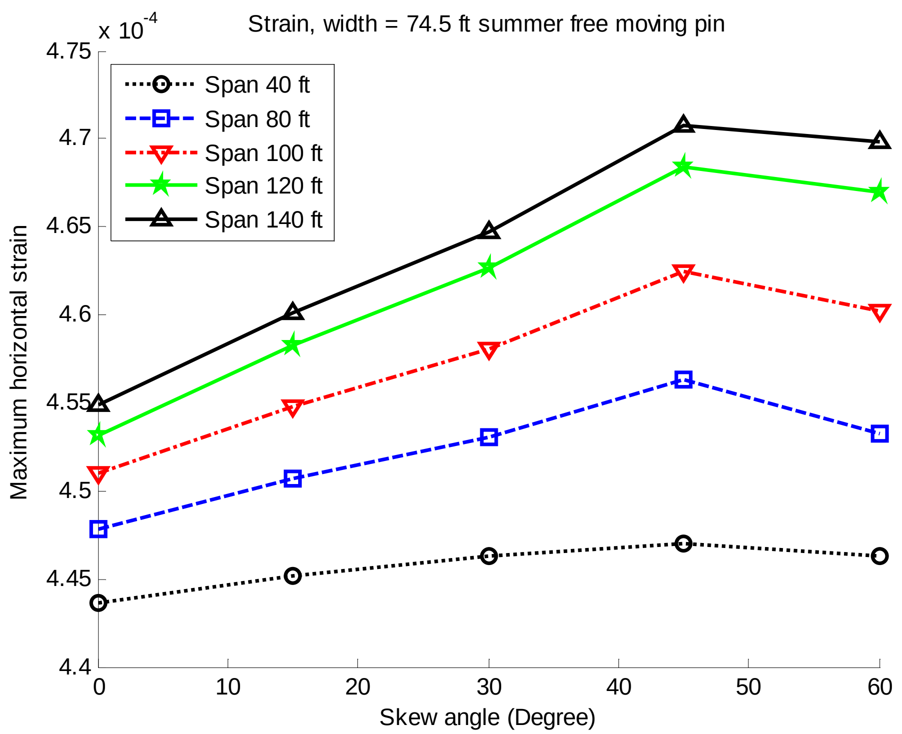


Figure E-33 Maximum strain of bridges under summer temperature (width = 74.5 ft, free moving pin and hanger)

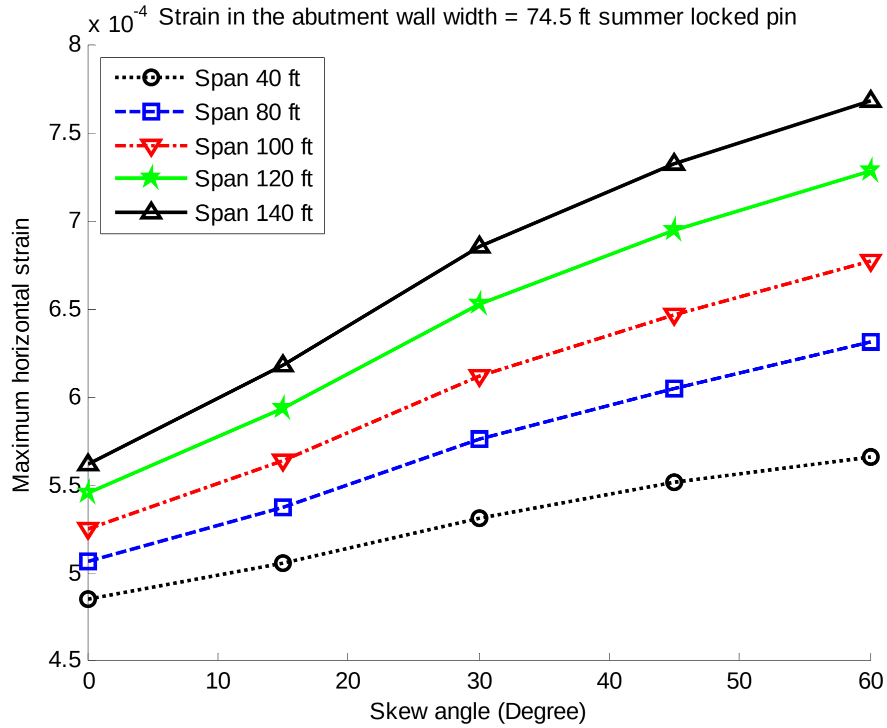


Figure E-34 Maximum strain of bridges under summer temperature (width = 42.5 ft, pin and hanger locked)

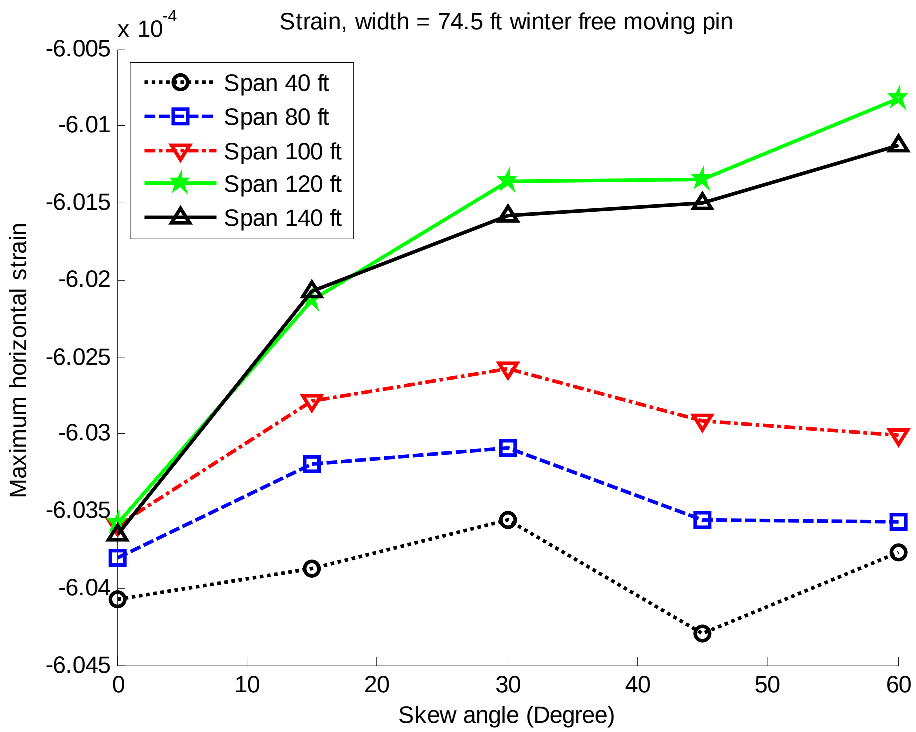


Figure E-35 Maximum strain of bridges under winter temperature (width = 42.5 ft, free moving pin and hanger)

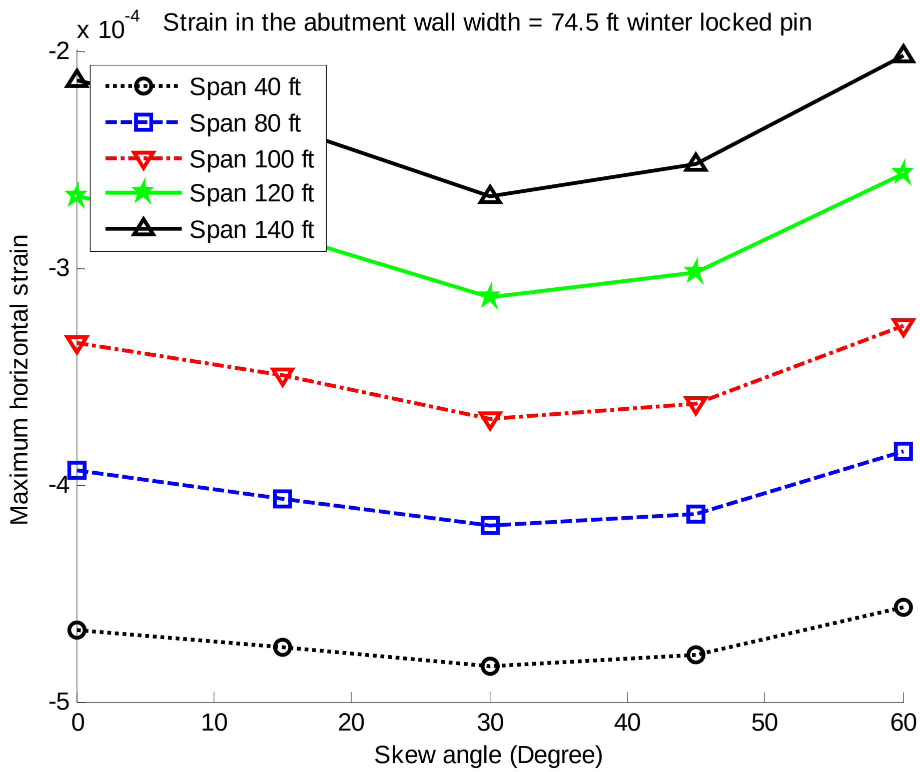


Figure E-36 Maximum strain of bridges under winter temperature (width = 42.5 ft, pin and hanger locked)

E.II Continuous Steel Bridges

The largest maximum principal stresses and the largest horizontal strains in the specified region of abutment walls of continuous steel bridges were shown in this section, Figure E-37 to **Figure** E-42 show bridges with deck width of 42.5 ft, **Figure** E-43 to **Figure** E-48 and **Figure** E-49 to **Figure** E-54 show bridges with deck width of 58.5 ft and 74.5 ft; respectively.

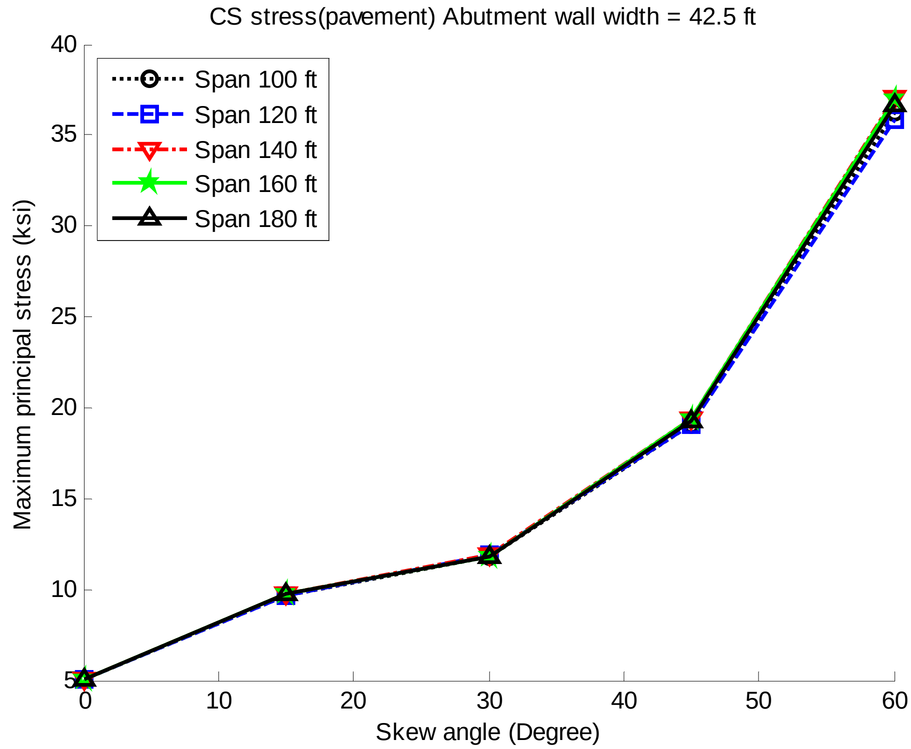


Figure E-37 Maximum stress of continuous steel bridges under pavement pressure (width = 42.5 ft)

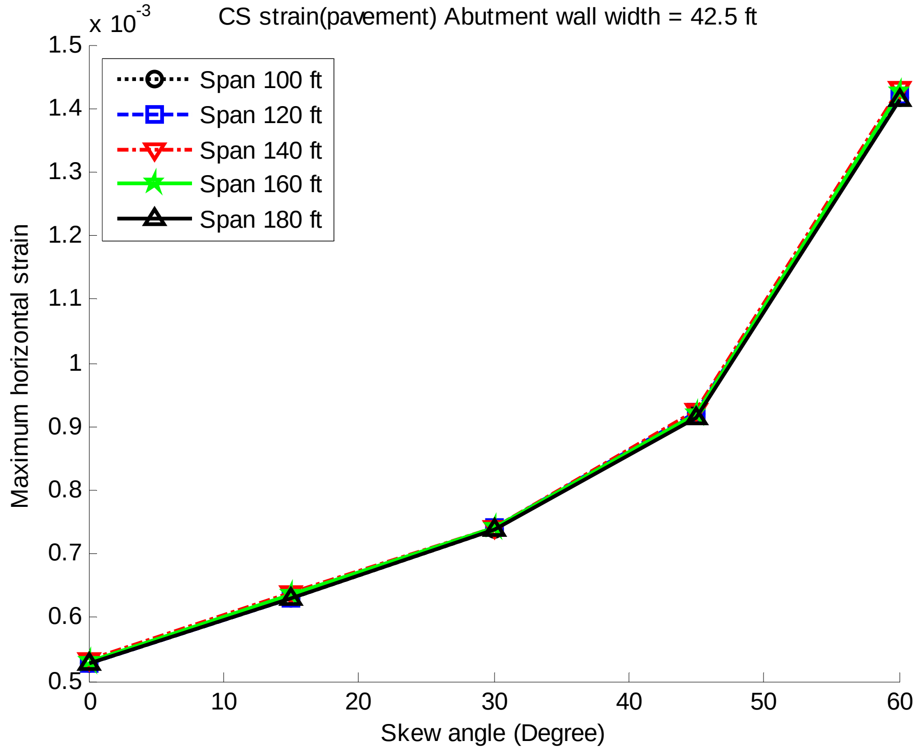


Figure E-38 Maximum strain of continuous steel bridges under pavement pressure (width = 42.5 ft)

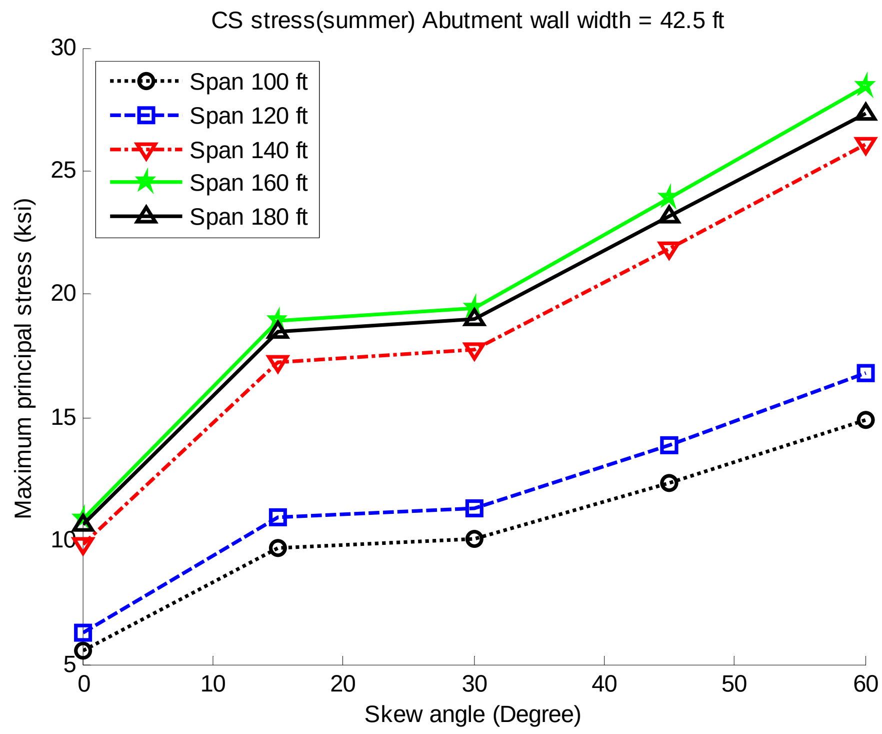


Figure E-39 Maximum stress of continuous steel bridges under summer temperature (width = 42.5 ft)

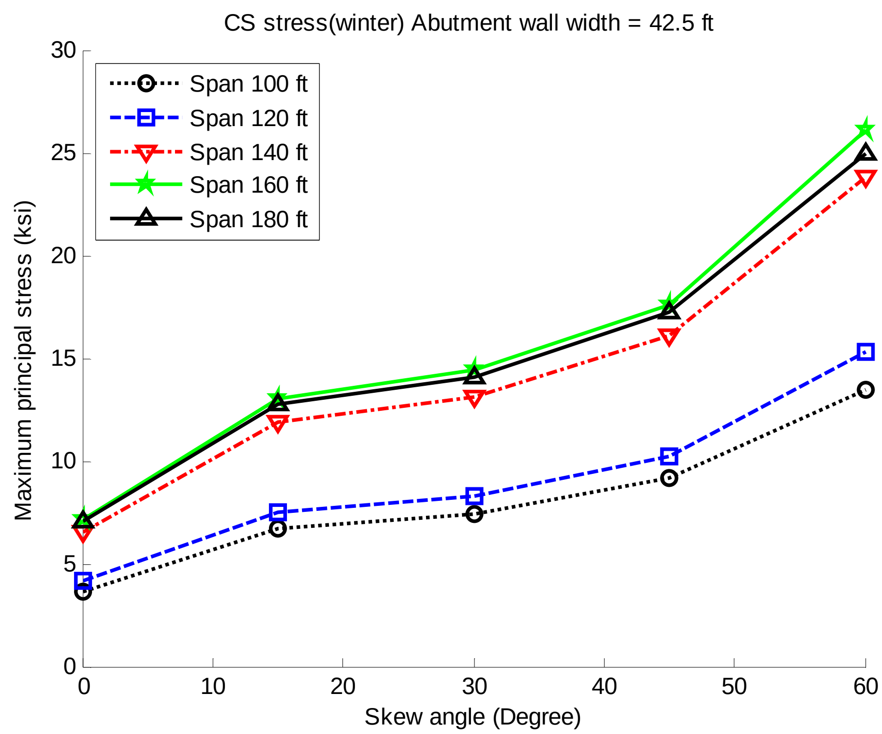


Figure E-40 Maximum stress of continuous steel bridges under winter temperature (width = 42.5 ft)

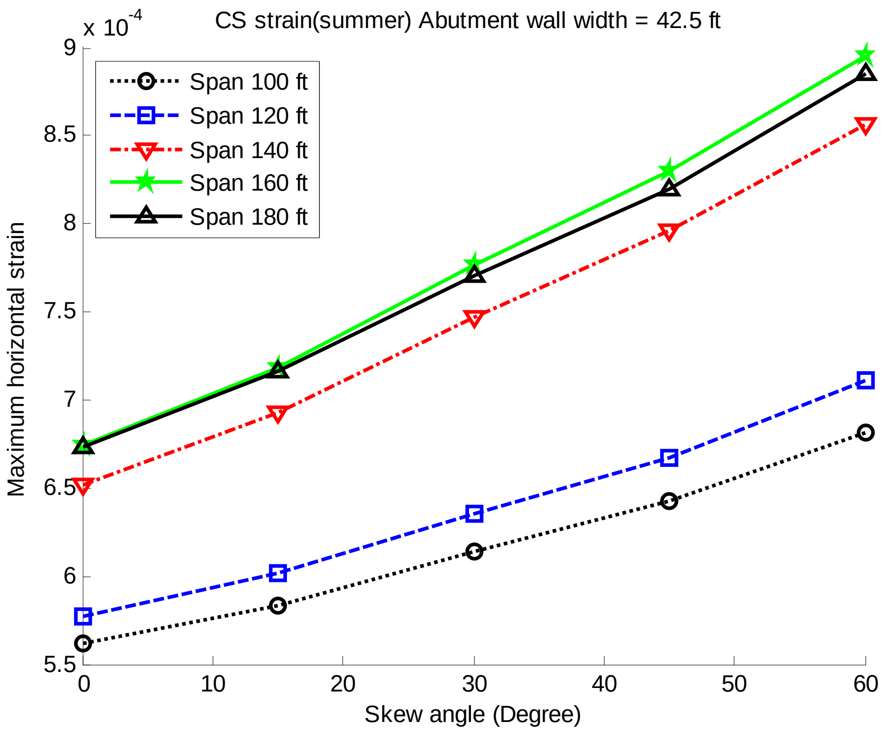


Figure E-41 Maximum strain of continuous steel bridges under summer temperature (width = 42.5 ft)

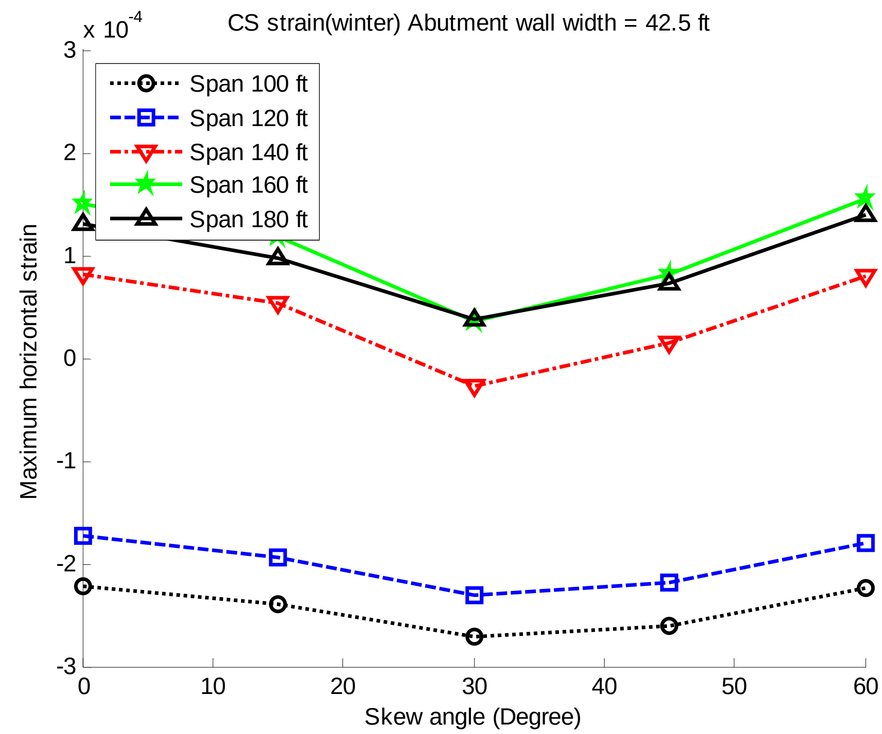


Figure E-42 Maximum strain of continuous steel bridges under winter temperature (width = 42.5 ft)

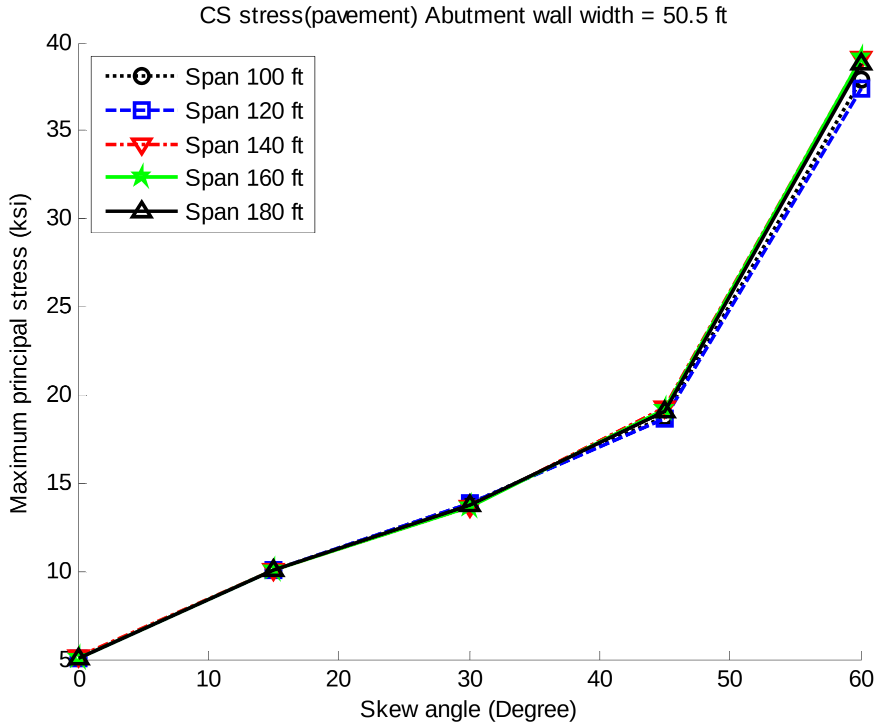


Figure E-43 Maximum stress of continuous steel bridges under pavement pressure (width = 50.5 ft)

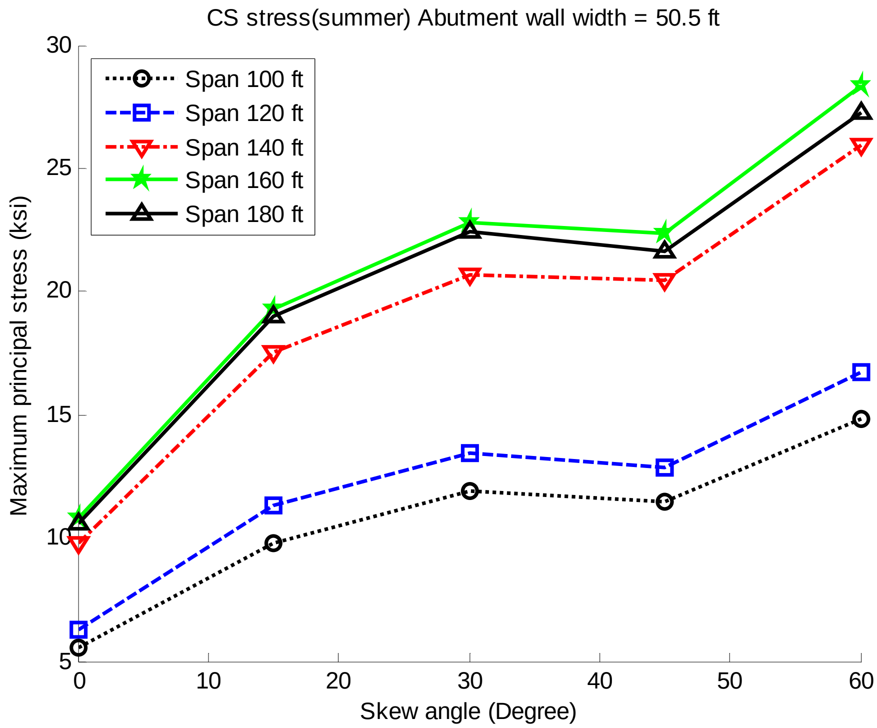


Figure E-44 Maximum stress of continuous steel bridges under summer temperature (width = 50.5 ft)

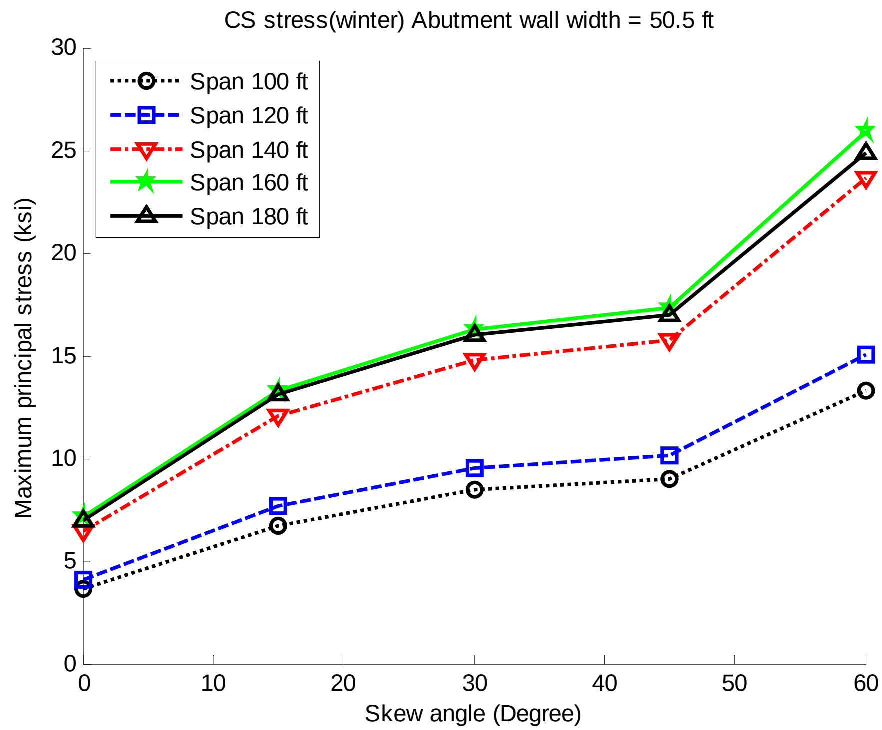


Figure E-45 Maximum stress of continuous steel bridges under winter temperature (width = 50.5 ft)

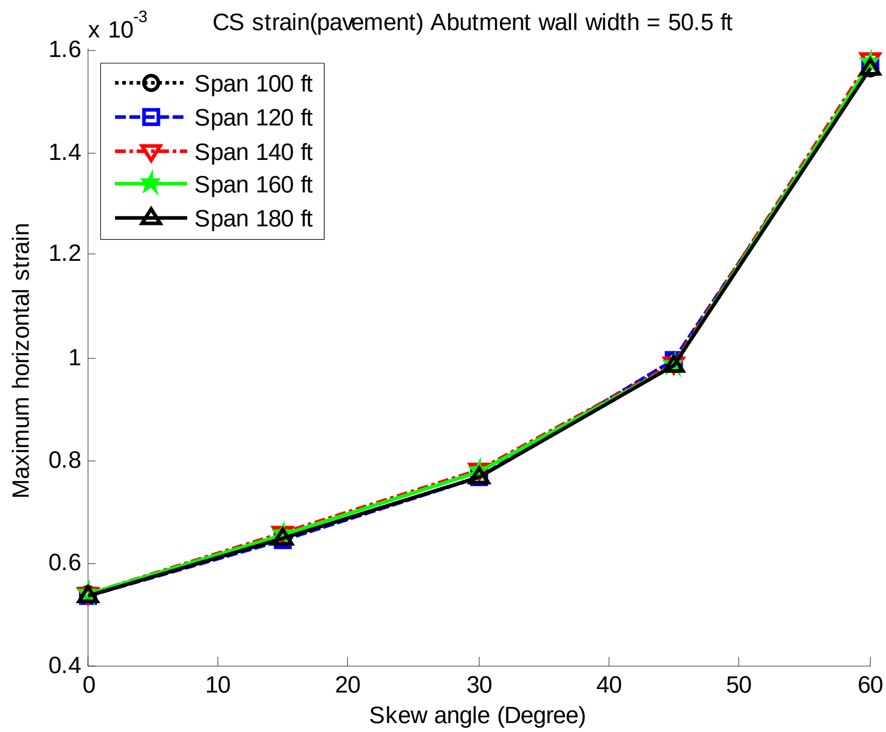


Figure E-46 Maximum strain of continuous steel bridges under pavement pressure (width = 50.5 ft)

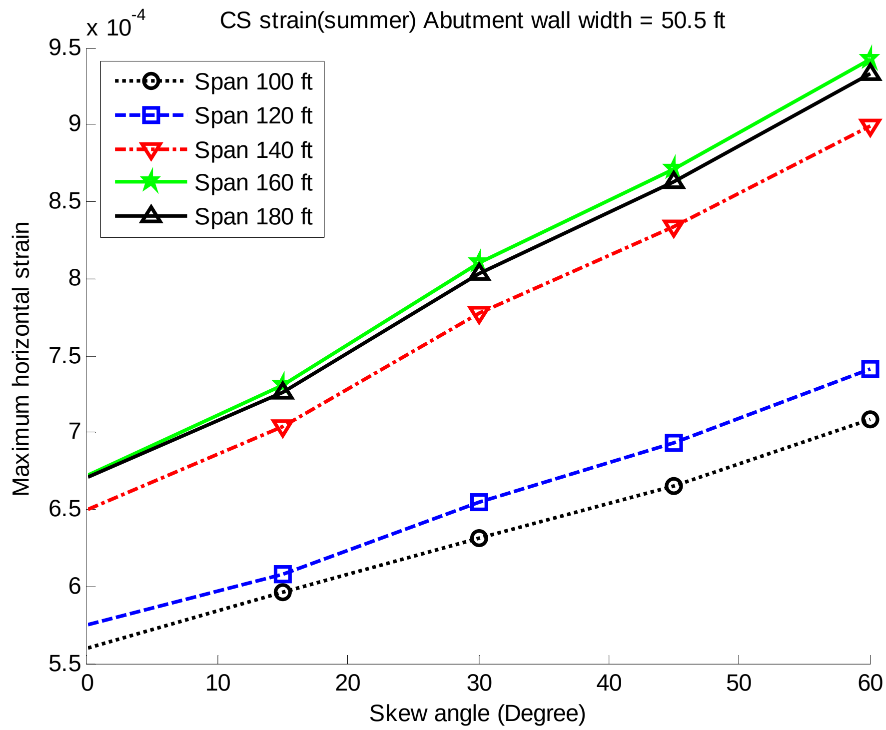


Figure E-47 Maximum strain of continuous steel bridges under summer temperature (width = 50.5 ft)

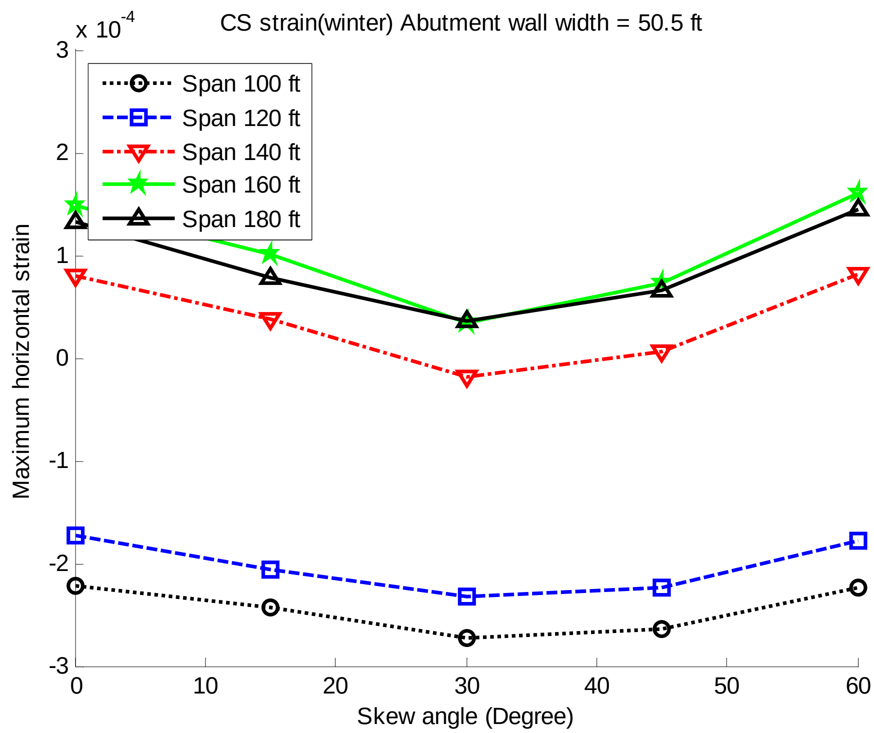


Figure E-48 Maximum strain of continuous steel bridges under winter temperature (width = 50.5 ft)

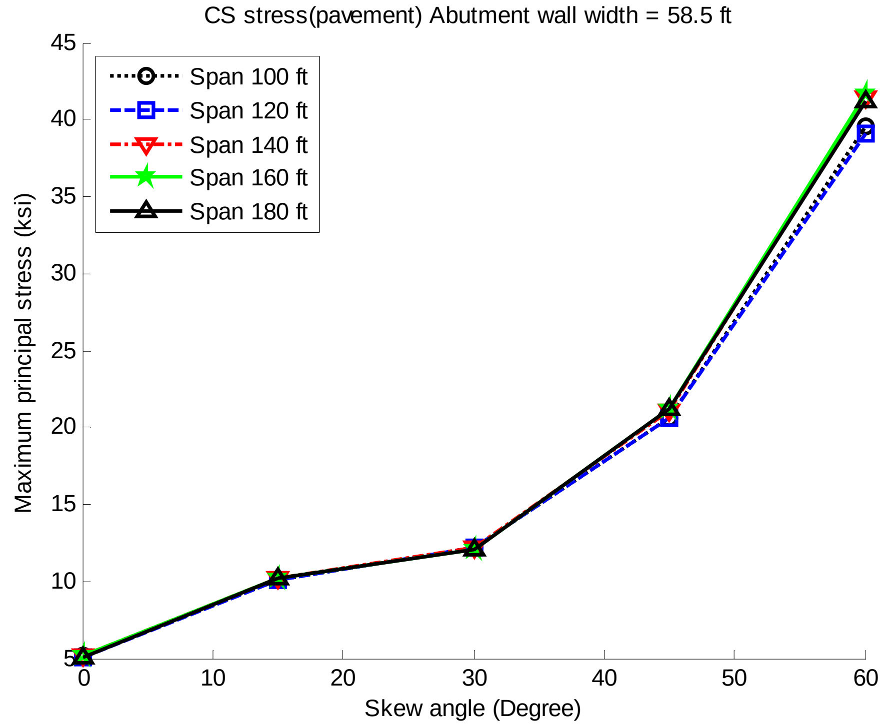


Figure E-49 Maximum stress of continuous steel bridges under pavement pressure (width = 58.5 ft)

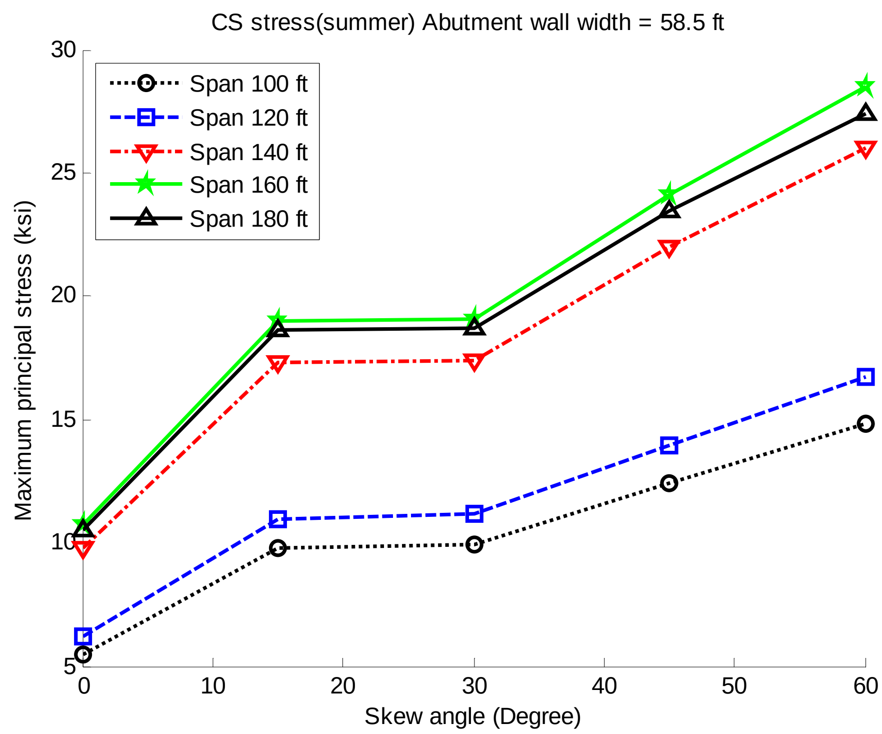


Figure E-50 Maximum stress of continuous steel bridges under summer temperature (width = 58.5 ft)

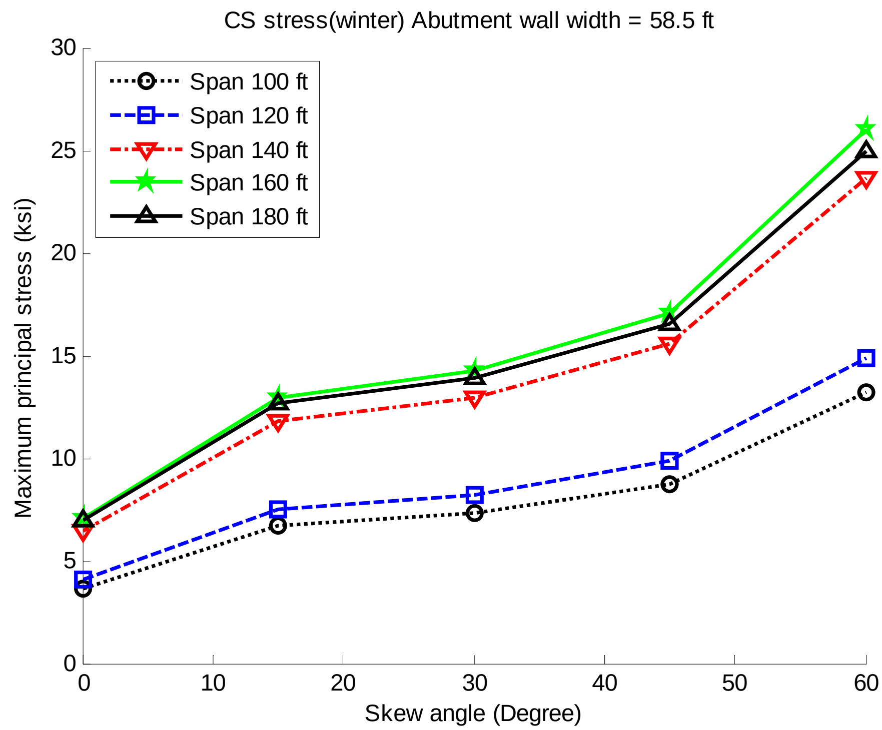


Figure E-51 Maximum stress of continuous steel bridges under winter temperature (width = 58.5 ft)

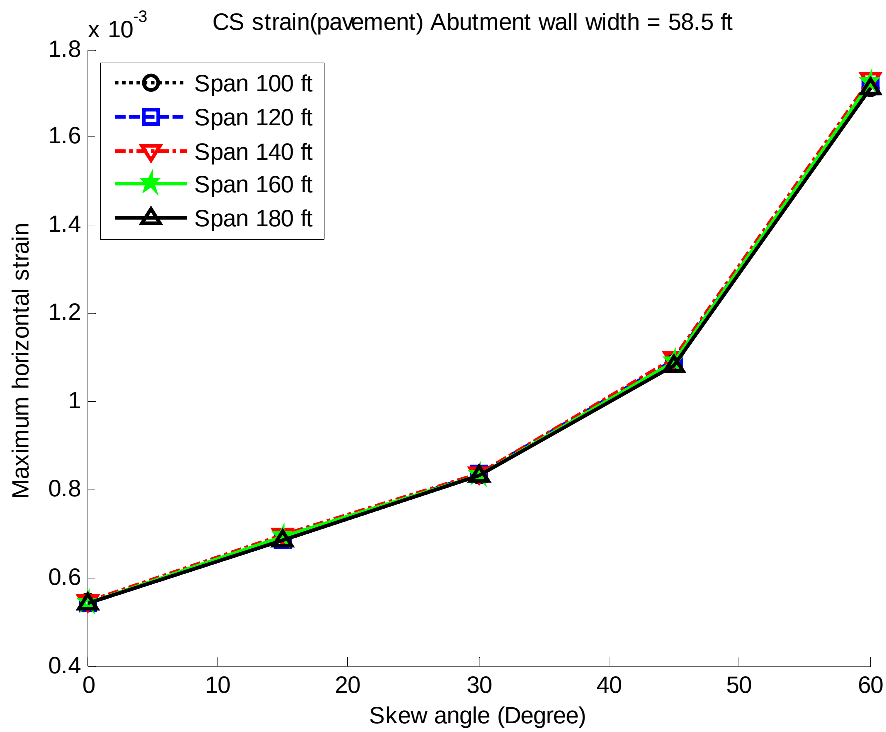


Figure E-52 Maximum strain of continuous steel bridges under pavement pressure (width = 58.5 ft)

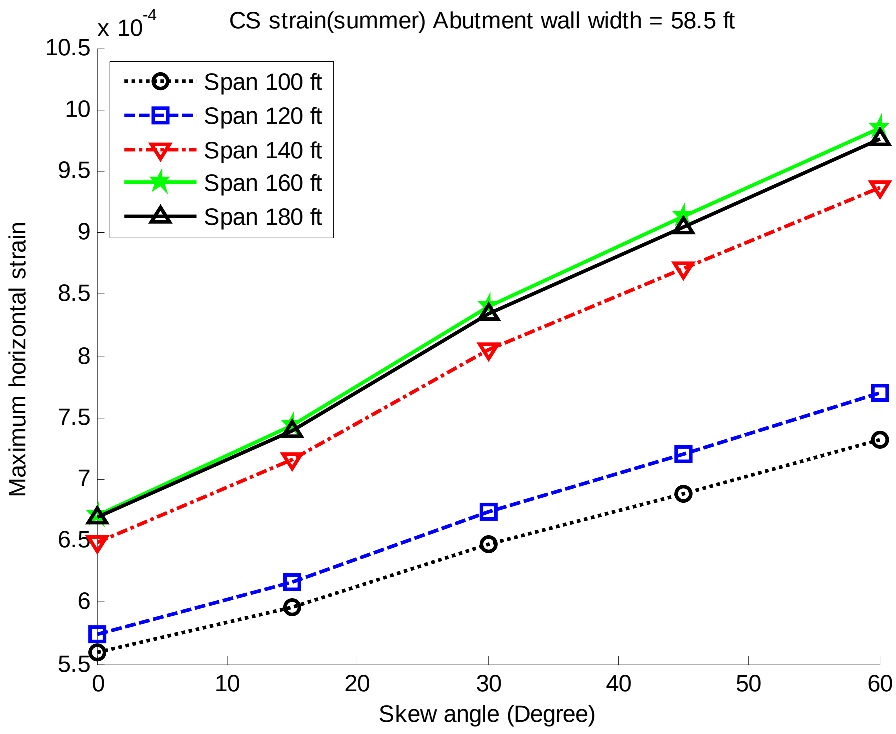


Figure E-53 Maximum strain of continuous steel bridges under summer temperature (width = 58.5 ft)

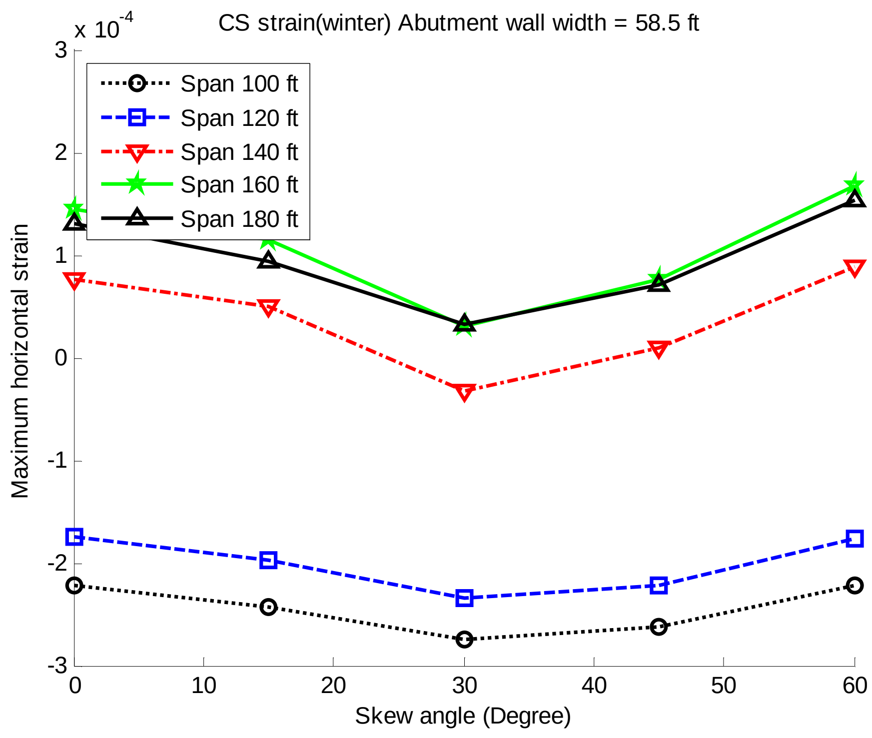


Figure E-54 Maximum strain of continuous steel bridges under winter temperature (width = 58.5 ft)

E.III Prestressed Concrete Bridges with I girder

The largest maximum principal stresses and the largest horizontal strains in the specified region of abutment walls of prestressed concrete bridges were shown in this section, Figure E-55 to **Figure** E-60 show bridges with deck width of 42.5 ft, **Figure** E-61 to **Figure** E-66 and **Figure** E-67 to **Figure** E-72 show bridges with deck width of 58.5 ft and 74.5 ft; respectively.

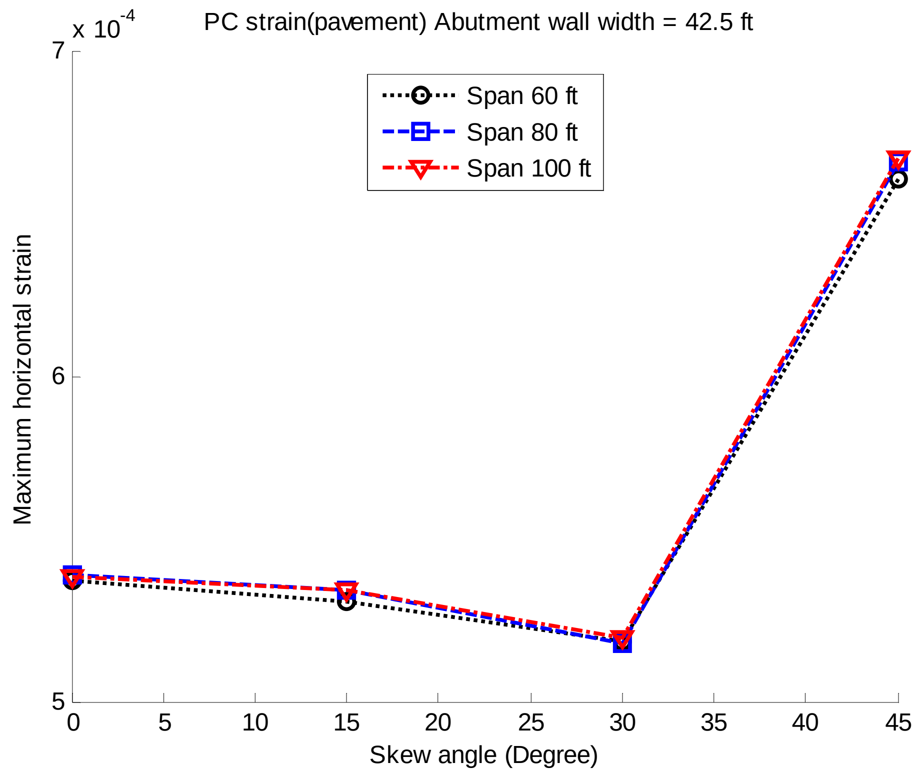


Figure E-55 Maximum strain of prestressed concrete bridges under pavement pressure (width = 42.5 ft)

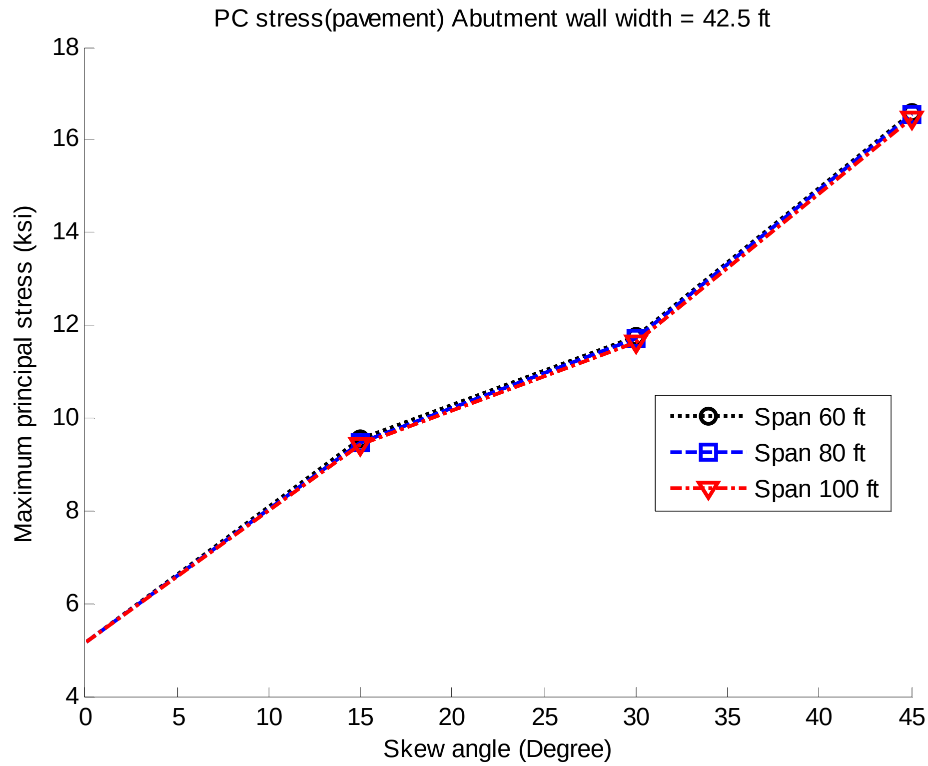


Figure E-56 Maximum stress of prestressed concrete bridges under pavement pressure (width = 42.5 ft)

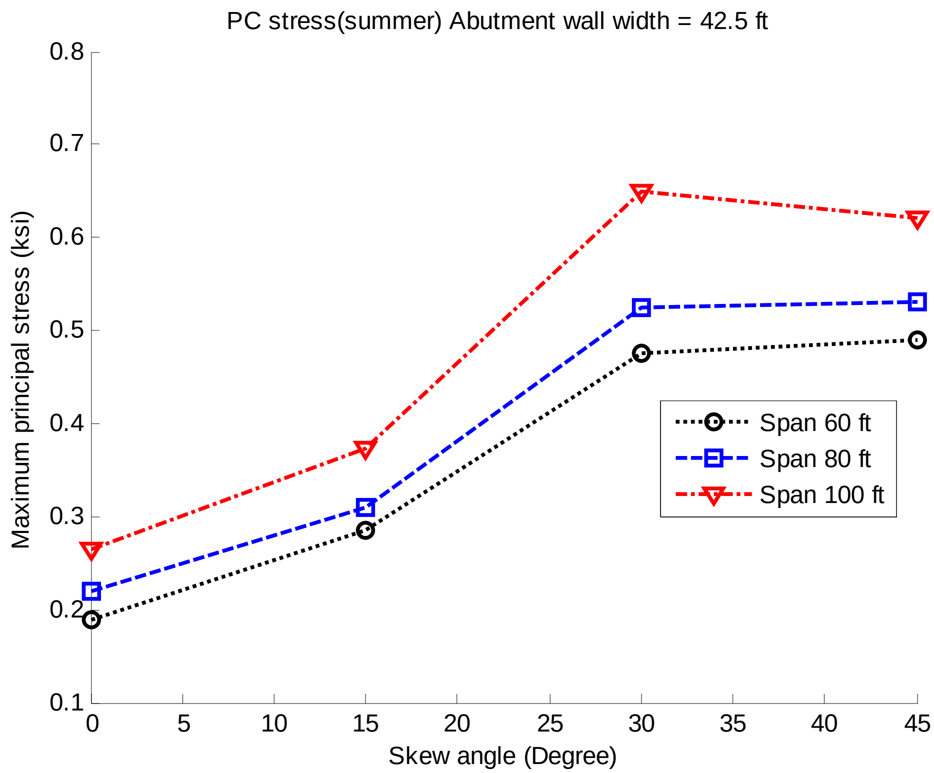


Figure E-57 Maximum stress of prestressed concrete bridges under summer temperature (width = 42.5 ft)

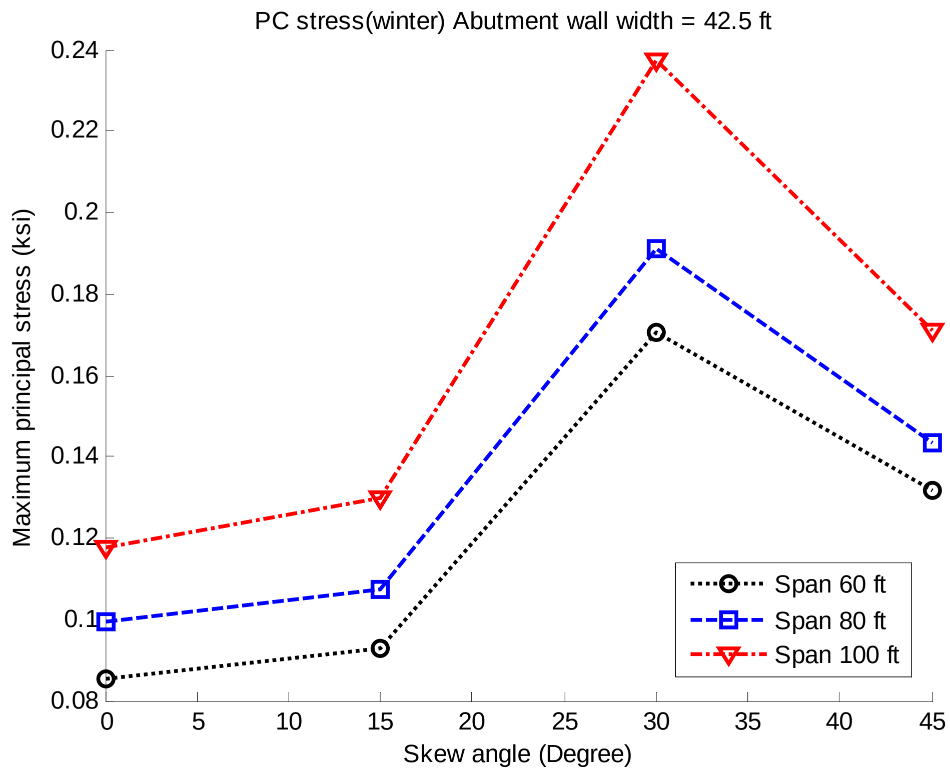


Figure E-58 Maximum stress of prestressed concrete bridges under winter temperature (width = 42.5 ft)

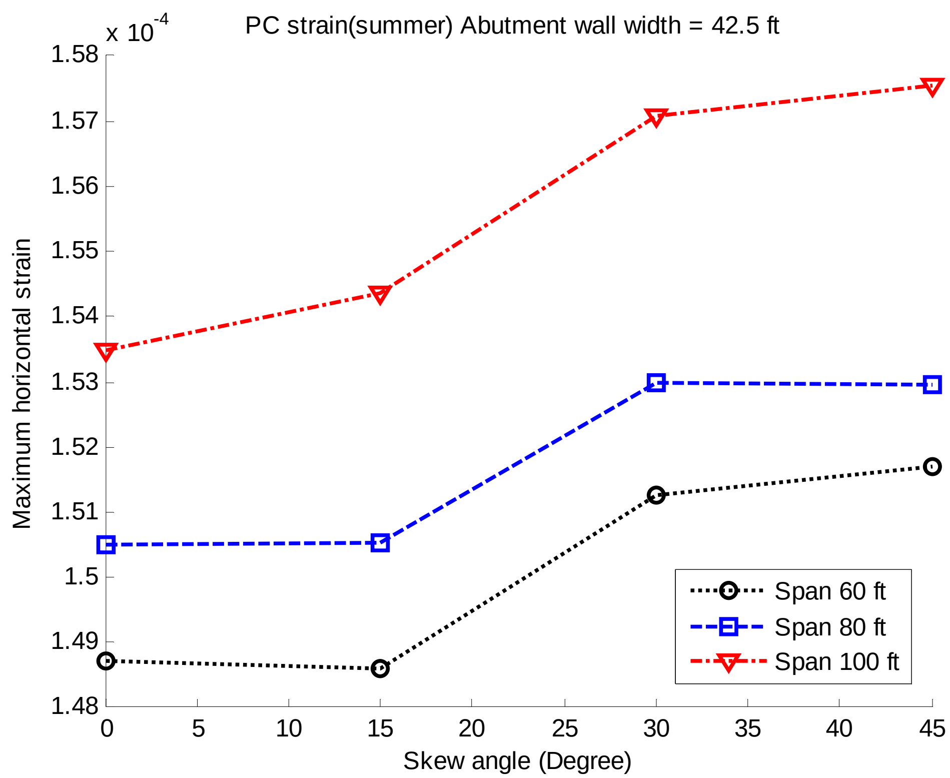


Figure E-59 Maximum strain of prestressed concrete bridges under summer temperature (width = 42.5 ft)

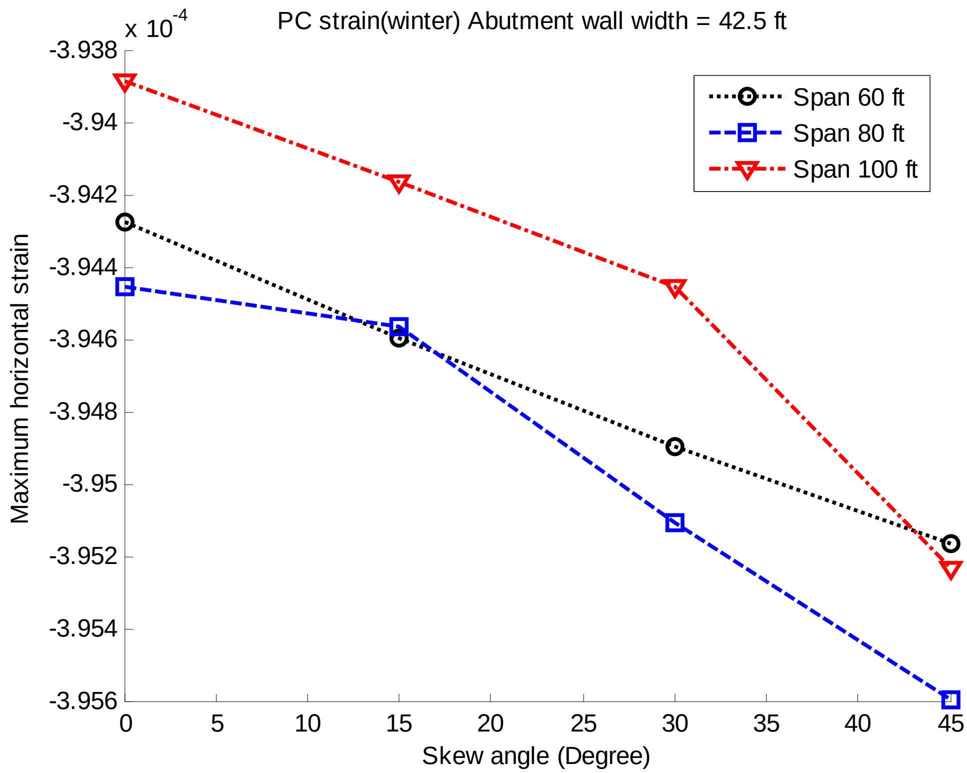


Figure E-60 Maximum strain of prestressed concrete bridges under winter temperature (width = 42.5 ft)

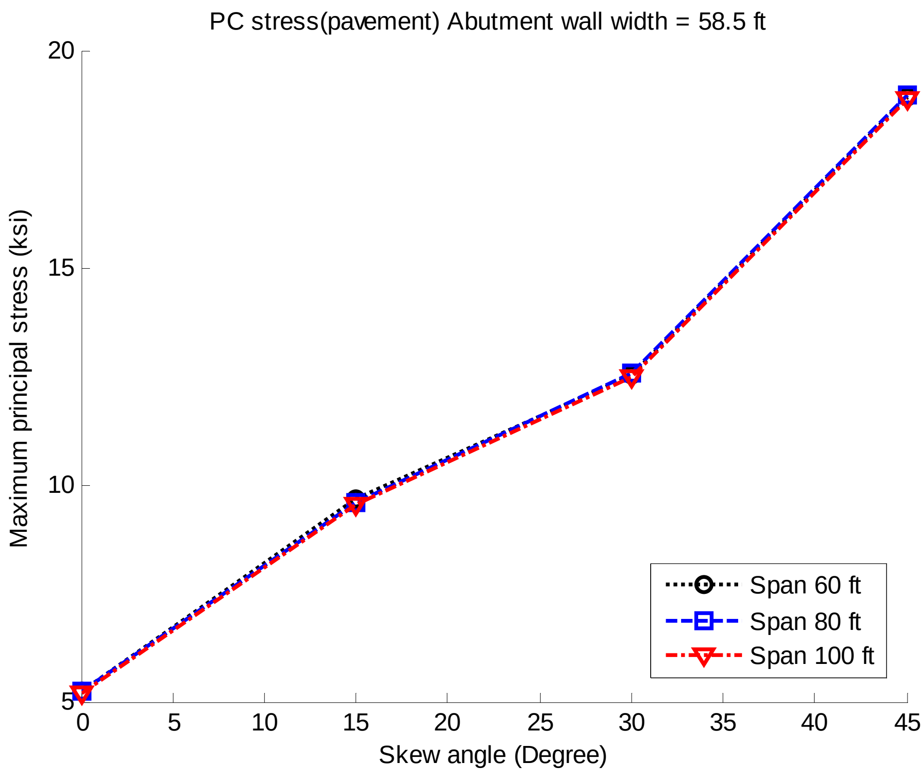


Figure E-61 Maximum stress of prestressed concrete bridges under pavement pressure (width = 58.5 ft)

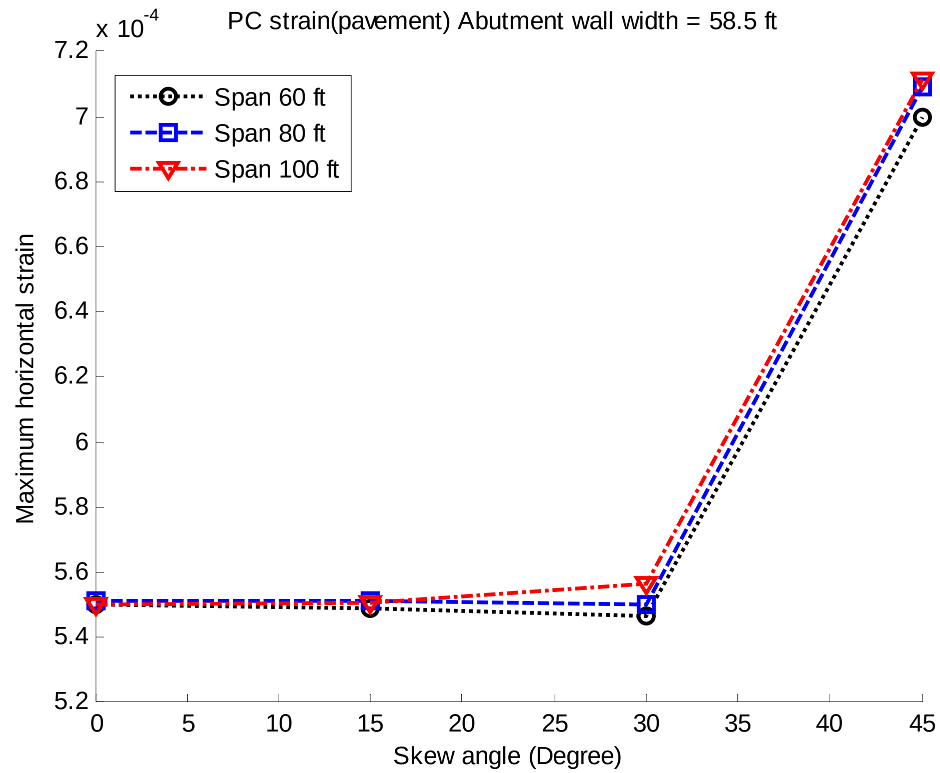


Figure E-62 Maximum strain of prestressed concrete bridges under pavement pressure (width = 58.5 ft)

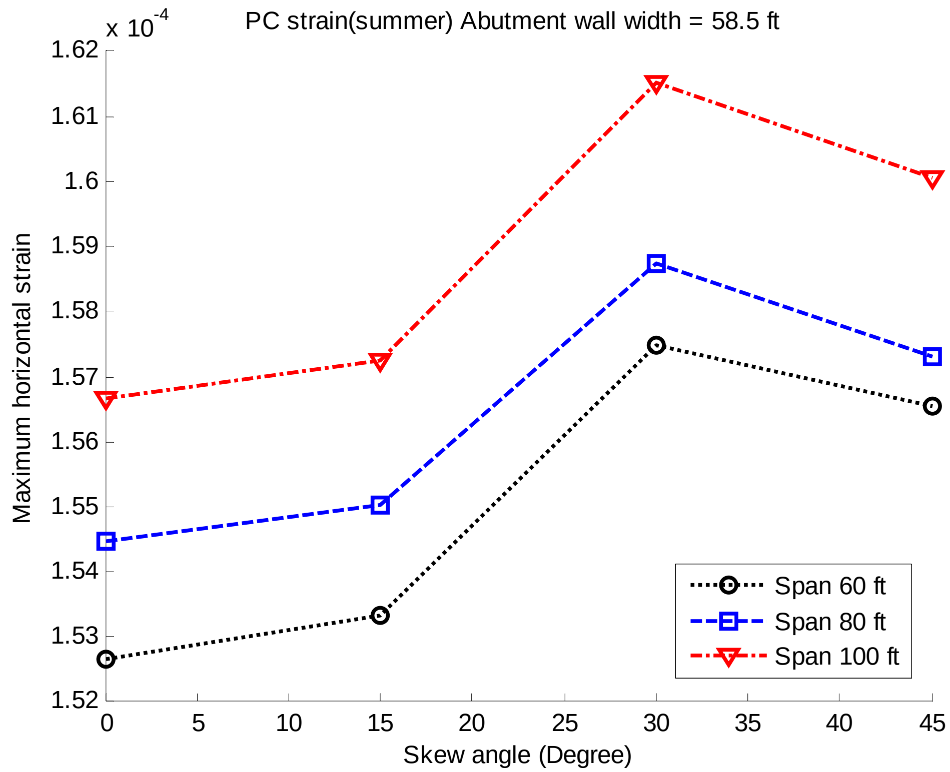


Figure E-63 Maximum strain of prestressed concrete bridges under summer temperature (width = 58.5 ft)

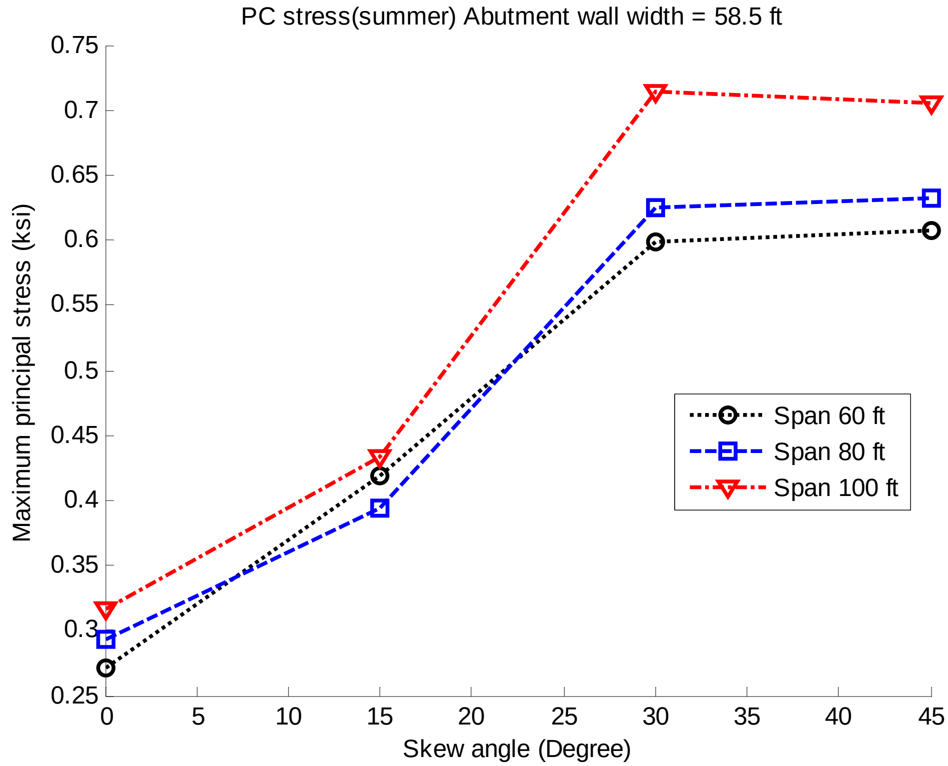


Figure E-64 Maximum stress of prestressed concrete bridges under summer temperature (width = 58.5 ft)

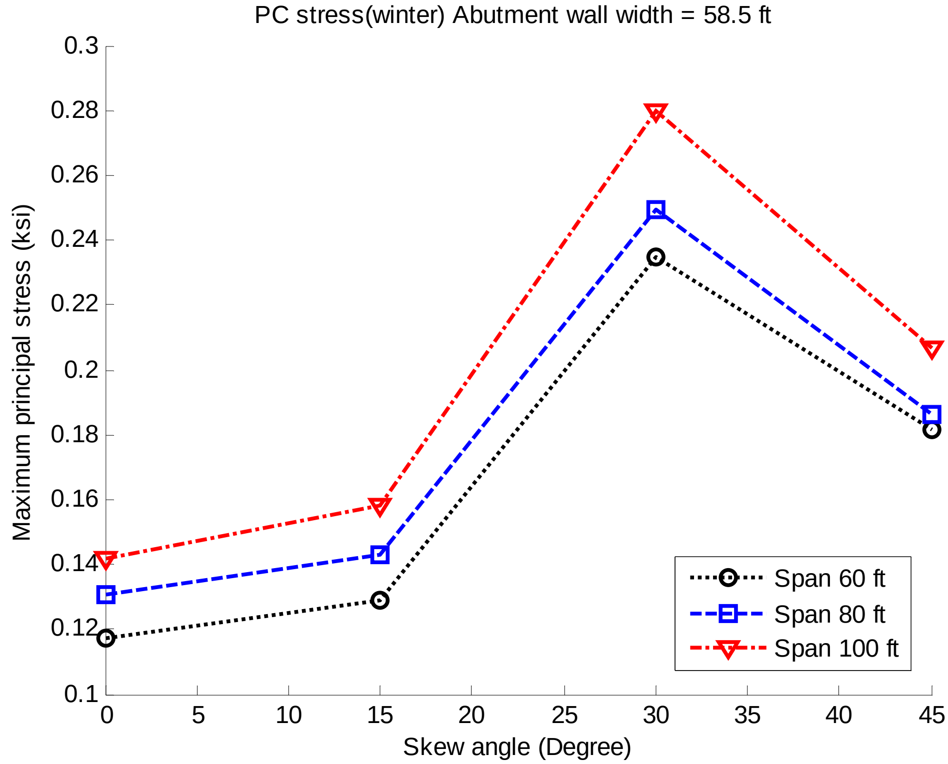


Figure E-65 Maximum stress of prestressed concrete bridges under winter temperature (width = 58.5 ft)

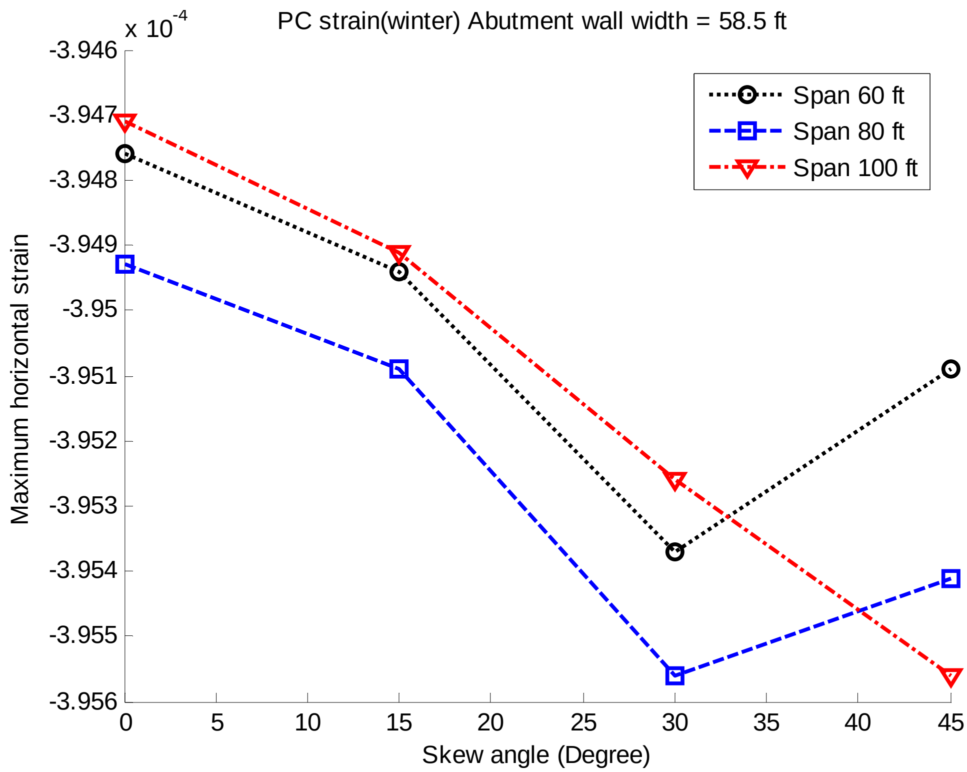


Figure E-66 Maximum strain of prestressed concrete bridges under winter temperature (width = 58.5 ft)

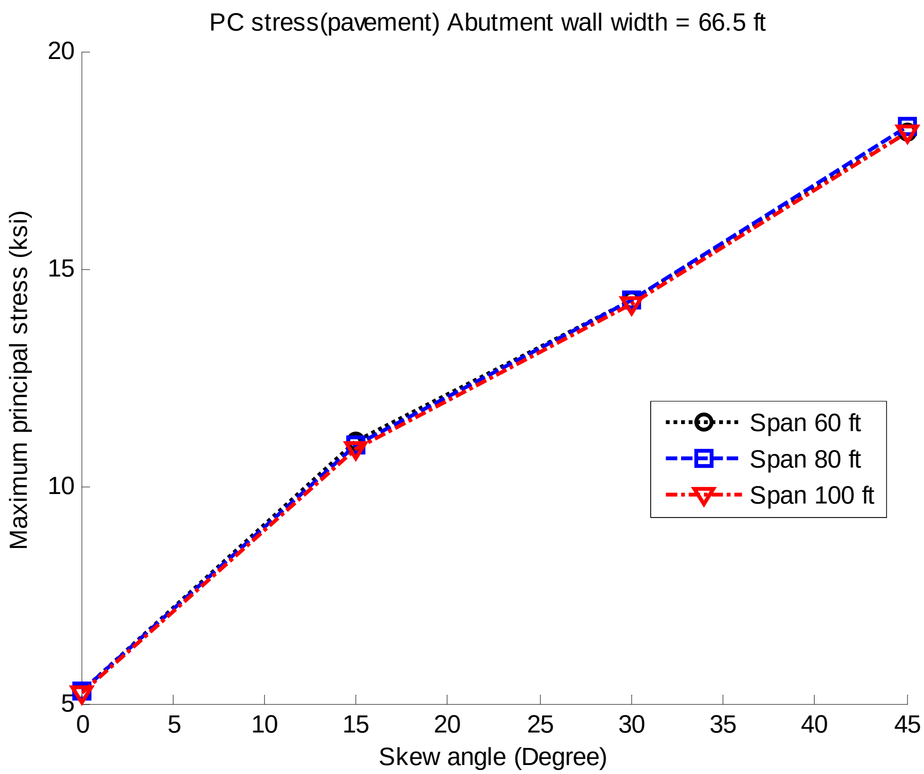


Figure E-67 Maximum stress of prestressed concrete bridges under pavement pressure (width = 66.5 ft)

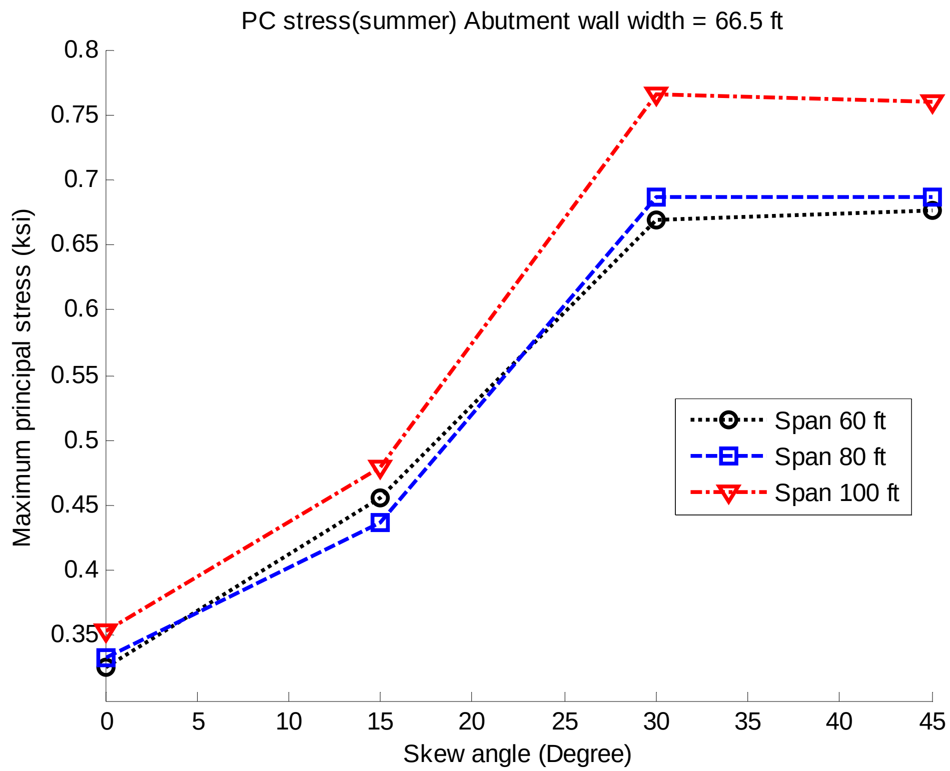


Figure E-68 Maximum stress of prestressed concrete bridges under summer temperature (width = 66.5 ft)

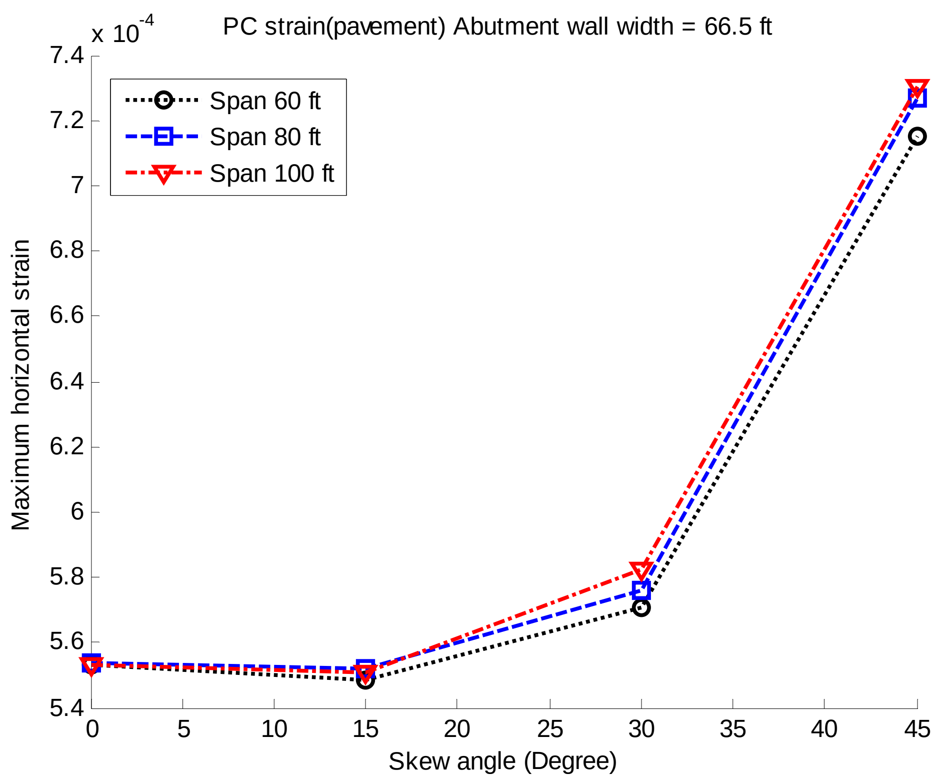


Figure E-69 Maximum strain of prestressed concrete bridges under pavement pressure (width = 66.5 ft)

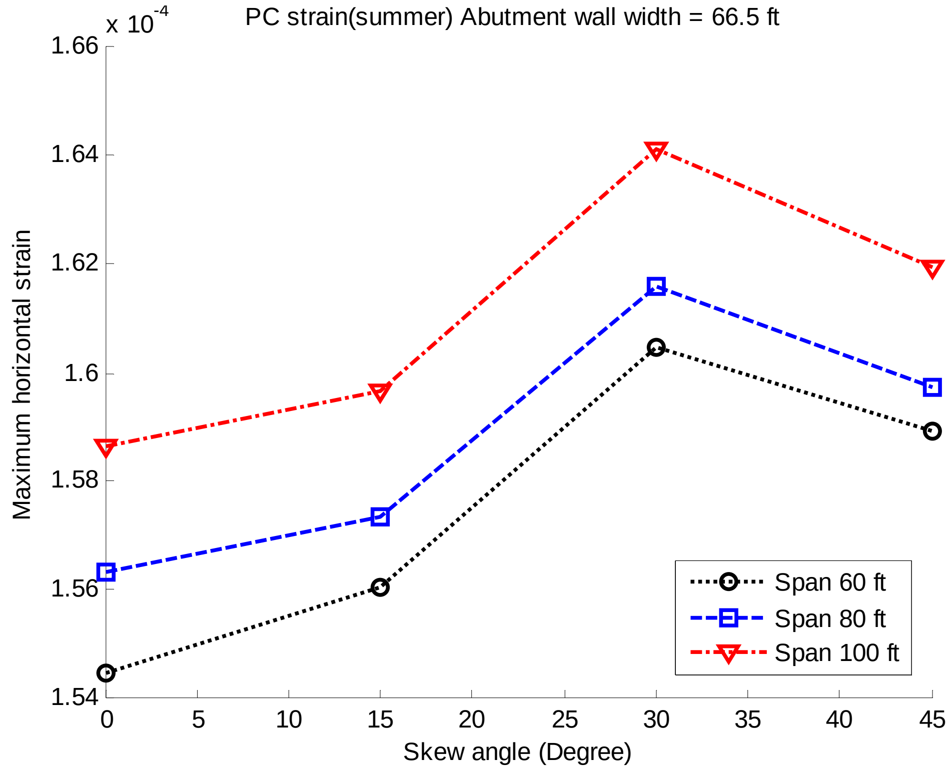


Figure E-70 Maximum strain of prestressed concrete bridges under summer temperature (width = 66.5 ft)

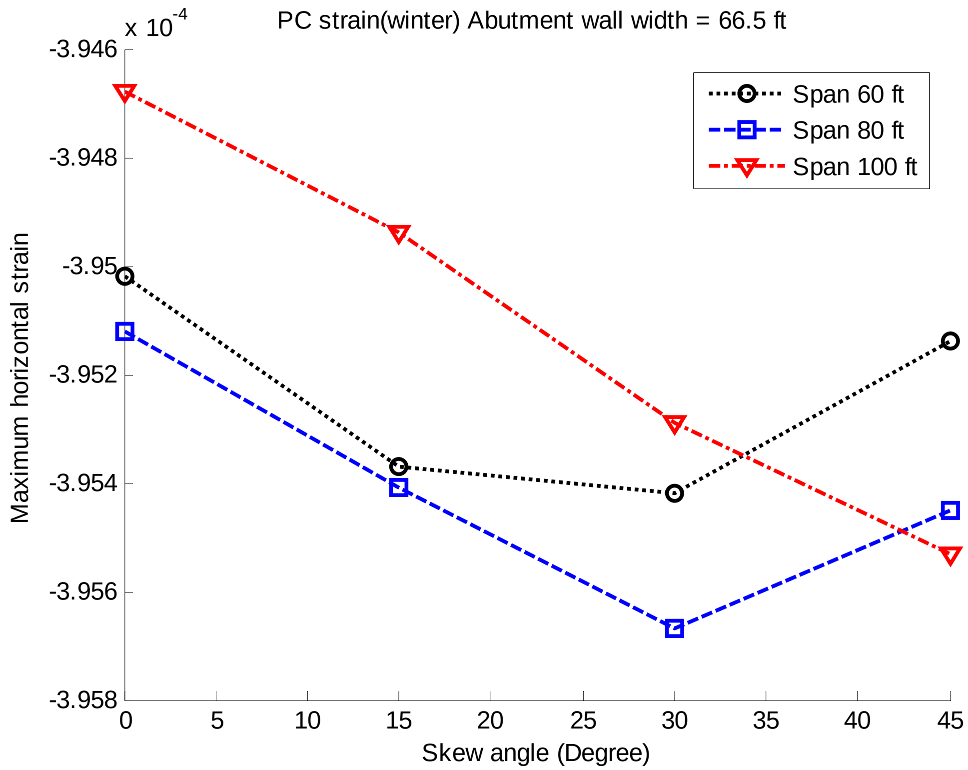


Figure E-71 Maximum strain of prestressed concrete bridges under winter temperature (width = 66.5 ft)

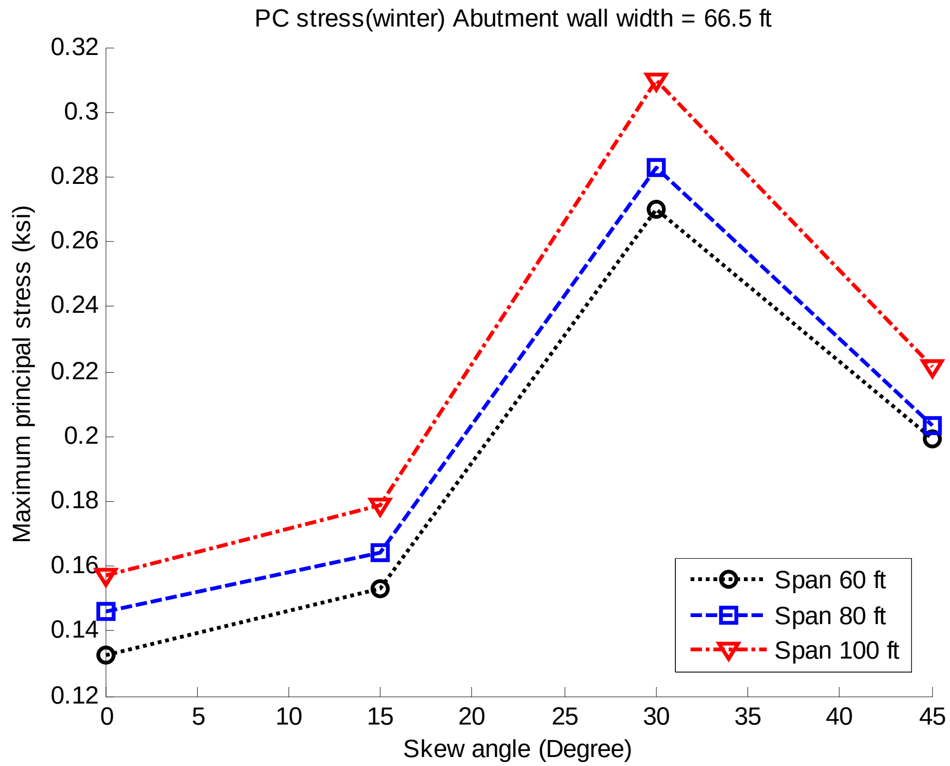


Figure E-72 Maximum stress of prestressed concrete bridges under winter temperature (width = 66.5 ft)

F. Bridge Abutment Diagnosis (SbNET) 1.2 User's Manual

F.I Introduction

Bridge Abutment Damage Diagnosis (SbNET 1.2) is a program developed by Michigan State University for the Michigan Department of Transportation (MDOT) as part of a research project to determine the causes and provide methods to alleviate the damage of bridge abutment walls. SbNET is a stand-along executable compiled from Matlab (Mathworks 2007) codes using a pre-trained ensemble of networks described in Section 6.5 of the final research report (Burgueño and Li 2008). SbNET can make predictions for new bridges using design parameters or for existing bridges using the MDOT Bridge ID. The program estimates the bridge abutment rating given the age of the bridge and provides a deterioration curve for the bridge service life. Output can be saved in text files for further post-processing.

F.II Installation

F.II.1 Copy CD files to destination folder

Copy the files in the CD to the location where SbNET is to be executed.

F.II.2 Set up MCRInstaller.

Install the Matlab Component Runtime (MCR) through the MCRInstaller. You need to have administrative rights in the computer/account in order to do this installation. Double click to open MCRInstaller and follow the InstallShield Wizard instructions. When prompted for the path to install **Matlab Component Runtime 7.6**, provide the following:

C:\ProgramFiles\MATLAB\MATLAB Component Runtime\

After finishing the installation, the MCRInstaller file may be deleted.

F.II.3 Executing SbNET 1.2

SbNET12 can be run after the installation of MCR by double-clicking the sb12 console icon. A shortcut to launch SbNET12 can be created on the desktop by following steps: on the desktop right click mouse—New—Shortcut—Browse—select sb12 in the corresponding folder—OK—Next—Finish.

F.III User Interface

The user interface of SbNET and how to use it will be illustrated using two examples.

F.III.1 Predict Using MDOT Bridge ID

The first example is using MDOT bridge id as input, as shown in Figure F-1 and Figure F-2.

Step 1: Choose the Input Method

The first step is choosing input method: input 1 if you would like to predict the deterioration curve for an existing MDOT highway bridge by input the bridge ID, refer to Figure F-1.

Step 2 Input Bridge ID

Quote mark needs to be included; the format needs to conform to Michigan Structure Inventory and Appraisal Coding Guide and capital "S" must be used. An example input for this step is: ‘82182053000S020’, as shown in Figure F-1.

SbNET will retrieve bridge information from the database and then predict the abutment condition in the future and simulate the abutment condition in the past. The discrete integer prediction values will be fitted to a smooth logistic curve. The exact prediction of bridge abutment condition during 70 years of bridge life will be saved in the file "Exact_Prediction.txt", the fitted prediction values will be saved in the file "Fitted_Prediction.txt". In these two txt files, the first column is the age of bridge; the second column is the predicted abutment rating; the third column is the lower limit of the confidence band; and the fourth column is the upper limit of the confidence band.

Step 3 Options for Plotting Deterioration Curves

Three options are available in plotting the deterioration curves for the life cycle of bridges.

[1] The deterioration curve based on the integer, or exact, predictions, as shown in Figure F-3.

[2] The deterioration curve based on the smoothed predictions (Burgueño and Li 2008), as shown in Figure F-4.

[3] The deterioration curve based on both the exact and the smoothed predictions, as shown in Figure F-5.

“CB” in the legends of the figures means confidence band. The deterioration curves of the bridge will be saved an “emf” images automatically and be available for later use even after the program was closed. The location of those saved pictures is in the folder where the executable file was placed. The manual inspection ratings for that bridge will also be retrieved from the database and added to the plot. The software will retrieve current time from the computer and make prediction for the current rating of the abutment wall of the highway bridge.

It is also possible that the program calculates an age for current rating smaller than the ages for past manual inspections, as shown in Figure F-6. This is caused by the rebuilding of the bridge rather than a bug in the software.

Step 4 More Predictions?

You will be asked whether you would like to make prediction for another bridge; input 1 if you would, 2 if you would not like to make prediction for another bridge (Figure F-2).

```
********************************************************************************

                        WELCOME to USE SbNET 1.2

  --- A software in predicting abutment conditions of MDOT highway bridges.

********************************************************************************

--------------------------------------------------------------------------------
Please choose input method: bridge id or design parameters:

          [1] Bridge ID;
          [2] Bridge design parameters.
--------------------------------------------------------------------------------
:1

--------------------------------------------------------------------------------
Please input Bridge ID.
* Quote mark needs to be included, the format need to conform with Michigan
* Structure Inventory and Appraisal Coding Guide. Must use Capital "S".
--------------------------------------------------------------------------------
:'82182053000S020'

--------------------------------------------------------------------------------

Retrieving bridge information from database ...

--------------------------------------------------------------------------------
--------------------------------------------------------------------------------

Ensemble of networks is simulating the bridge abutment condition ...

--------------------------------------------------------------------------------

--------------------------------------------------------------------------------

The current abutment rating of the MDOT highway bridge is:
Abutment_Rating =

     4


--------------------------------------------------------------------------------

--------------------------------------------------------------------------------
Please input the type of the deterioration curve you would like to plot.
          [1] Deterioration curve based on exact predictions;
          [2] Deterioration curve based on smoothed predictions;
          [3] Deterioration curves based on both exact and smoothed predictions;
```

Figure F-1 Interface of SbNET 1.2 (predict using bridge ID)

```
--------------------------------------------------------------------------------
Please input the type of the deterioration curve you would like to plot.
          [1] Deterioration curve based on exact predictions;
          [2] Deterioration curve based on smoothed predictions;
          [3] Deterioration curves based on both exact and smoothed predictions;

--------------------------------------------------------------------------------
:3

============================================================================

*If the current prediction showed an age smaller than manual inspection
*records, it signified that the bridge had been rebuilt rather than a bug
*in the software.

============================================================================


----------------------------------------------------------------------------
The deterioration curve of the bridge in concern has been saved
in the folder as emf image
----------------------------------------------------------------------------
----------------------------------------------------------------------------
Would you like to plot another another type of deterioration curve?

          [1] Yes;
          [2] No.
----------------------------------------------------------------------------
:2
----------------------------------------------------------------------------
Would you like to make prediction for another birdge?

          [1] Yes;
          [2] No.
----------------------------------------------------------------------------
:
```

Figure F-2 Interface of SbNET 1.2_ continued (predict using bridge ID)

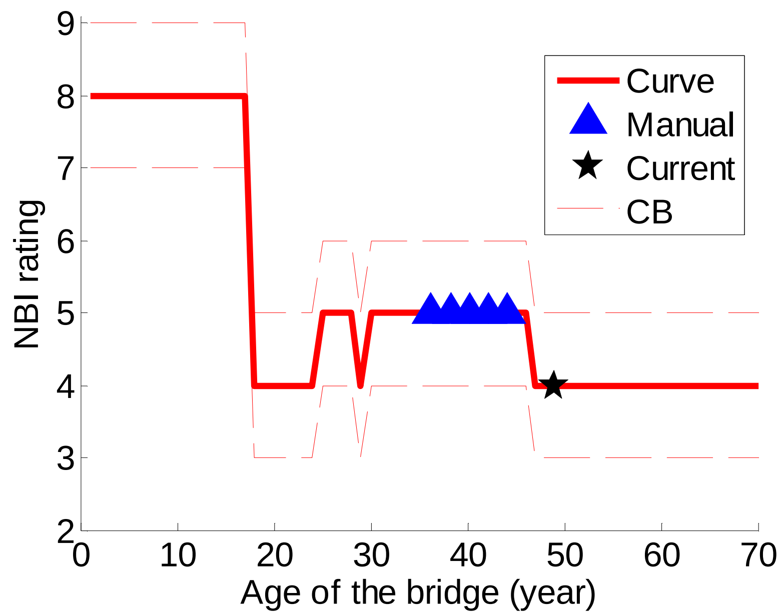


Figure F-3 An example of exact deterioration curve

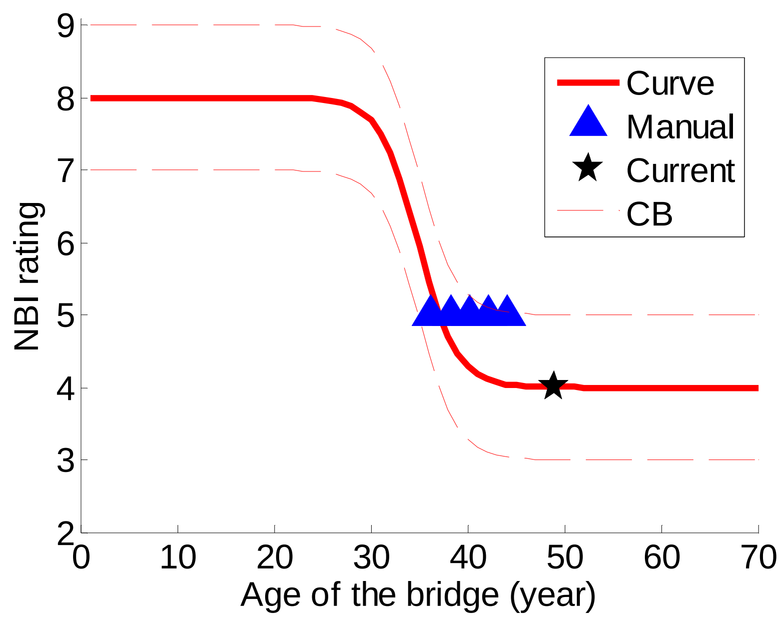


Figure F-4 An example of smoothed deterioration curve

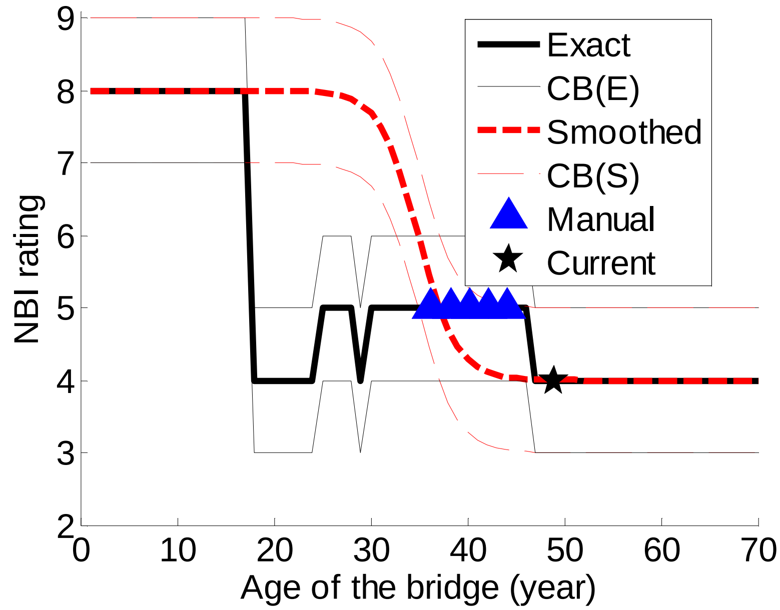


Figure F-5 An example of exact and smoothed deterioration curves

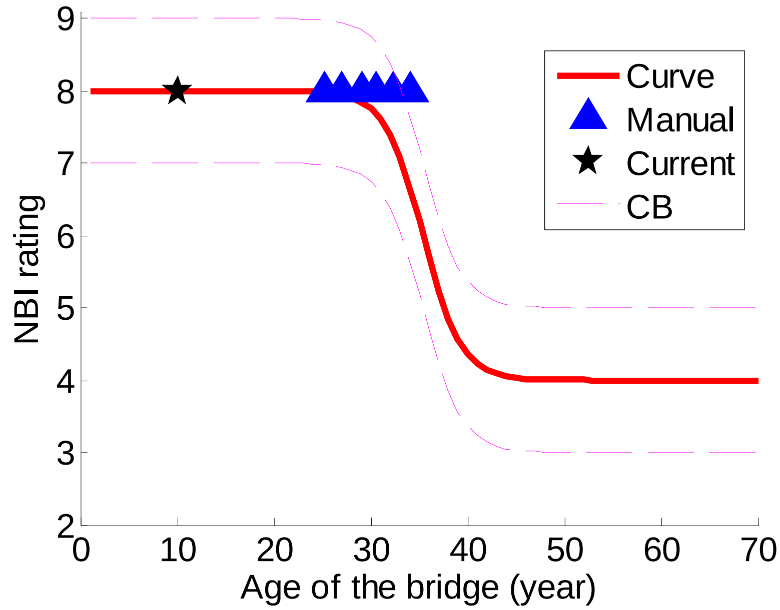


Figure F-6 An example of current prediction prior to manual inspections

F.III.2 Predict Using MDOT Bridge Design Parameters

The second example is using MDOT bridge design parameters as input, as shown in Figure F-7.

Step 1: Choose the input method

As shown in Figure F-7, the first step is choosing input method: input 2 if you would like to predict the deterioration of abutment wall for a highway bridge in design or an existing bridge which is not included in the refined database.

Step 2: Input bridge design parameters

Following design parameters need to be input through the interface, as shown in Figure F-7 and Figure F-8.

- Bridge length (unit: foot): bridge length needs to be in the range of 30 ft to 3281 ft.
- Skew angle (unit: degree): skew angle needs to be no less than 0 degree and less than 90 degree.
- Bridge width (unit: feet): Bridge width needs to be in the range of 20 ft to 100 ft.
- Age (unit: year): Age of a bridge needs to be in the range of 0 year to 80 years.
- Average daily truck traffic (ADTT unit: truck): ADTT is the average daily truck traffic volume for the inventory route. ADTT for a bridge shall be compatible with other recorded items. For example, if the bridge roadway widths are recorded separately for parallel bridges, then, ADTT also needs to be the separated recorded values. Please refer to Items 29 and 190 of Michigan Structure Inventory and Appraisal Coding Guide for details. In this software, ADTT value needs to be >= 100 & <= 30,000.
- State County code: State County code needs to conform to "Michigan Structure Inventory and Appraisal Coding Guide (page 9).
- Approach surface type: 1 for bitumen approach surface; 2 for mixed approach surface; 3 for concrete approach surface; and 4 for those bridges the approach surfaces are not known.

- Structural type of the bridge: 1 for simple/cantilevered steel bridge; 2 for continuous steel bridge; 3 for prestressed concrete bridge with I-girder; 4 for prestressed concrete bridge with adjacent box girder; and 5 for prestressed concrete bridge with spread box girder.

SbNET will predict the abutment condition of the bridge in 70 years’ of life. The discrete integer prediction values will be fitted to a smooth logistic curve. The exact prediction of bridge abutment condition during 70 years of bridge life will be saved in the file "Exact_Prediction.txt", the fitted prediction values will be saved in the file "Fitted_Prediction.txt". In these two txt files, the first column is the age of bridge; the second column is the predicted abutment rating; the third column is the lower limit of the confidence band; and the fourth column is the upper limit of the confidence band.

Step 3 Options for Plotting Deterioration Curves

Three options are available in plotting the deterioration curves for the life cycle of bridges.

[1] The deterioration curve based on the exact predictions, as shown in Figure F-9.

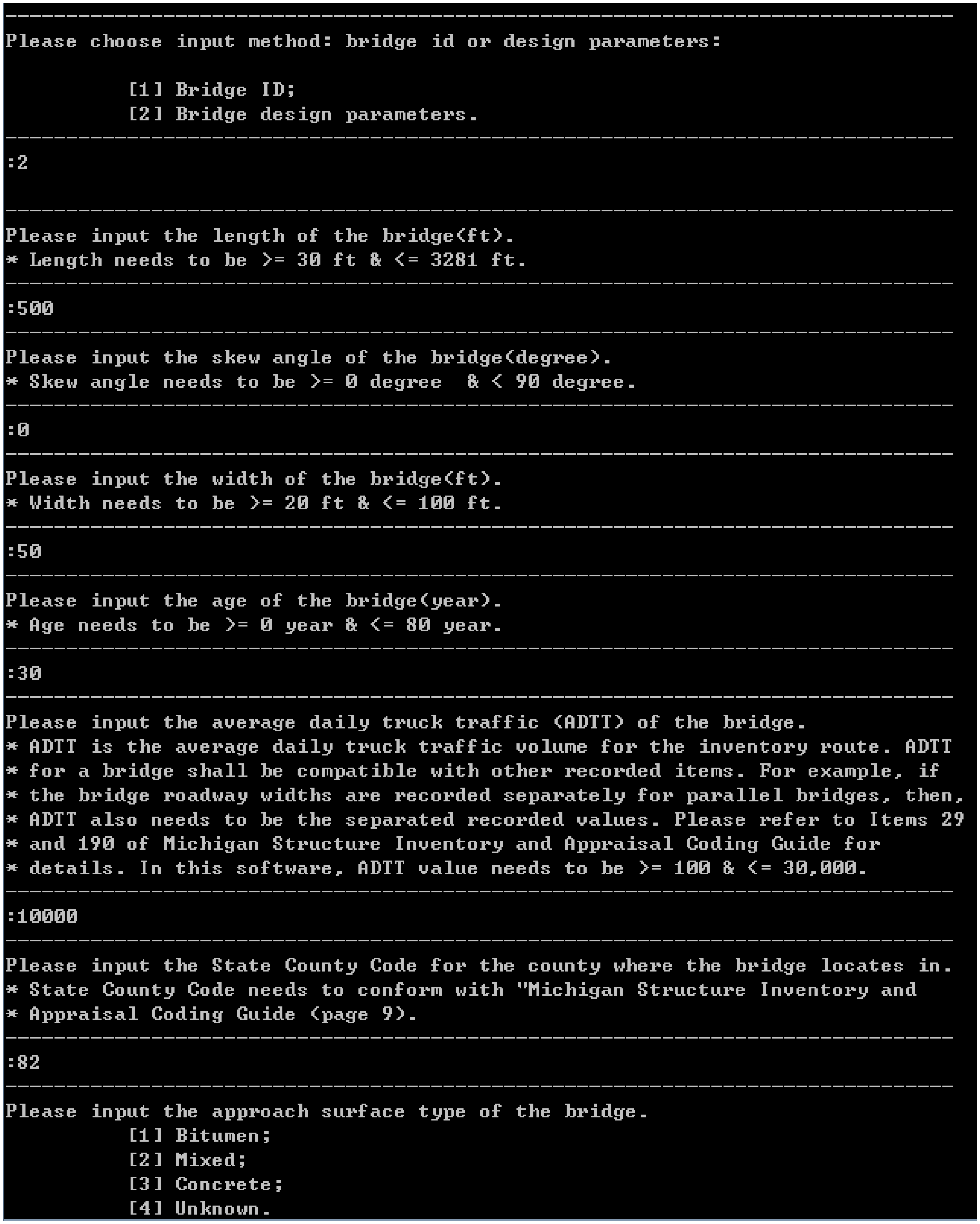
[2] The deterioration curve based on the smoothed predictions (Burgueño and Li 2008), as shown in Figure F-10.

[3] The deterioration curve based on both the exact and the smoothed predictions, as shown in Figure F-11.

“CB” in the legends of the figures means confidence band. The deterioration curves of the bridge will be saved an “emf” images automatically and be available for later use even after the program was closed. The location of those saved pictures is in the folder where the executable file was placed. The manual inspection ratings for that bridge will also be retrieved from the database and added to the plot. Based on the age input by the user, the software will also make prediction for the current rating of the abutment wall of the highway bridge.

Step 4 More Predictions?

You will be asked whether you would like to make prediction for another bridge (Figure F-8;) input 1 if you would, 2 if you would not like to make prediction for another bridge.



```
--------------------------------------------------------------------------------
Please choose input method: bridge id or design parameters:

          [1] Bridge ID;
          [2] Bridge design parameters.
--------------------------------------------------------------------------------
:2

--------------------------------------------------------------------------------
Please input the length of the bridge(ft).
* Length needs to be >= 30 ft & <= 3281 ft.
--------------------------------------------------------------------------------
:500
--------------------------------------------------------------------------------
Please input the skew angle of the bridge(degree).
* Skew angle needs to be >= 0 degree  & < 90 degree.
--------------------------------------------------------------------------------
:0
--------------------------------------------------------------------------------
Please input the width of the bridge(ft).
* Width needs to be >= 20 ft & <= 100 ft.
--------------------------------------------------------------------------------
:50
--------------------------------------------------------------------------------
Please input the age of the bridge(year).
* Age needs to be >= 0 year & <= 80 year.
--------------------------------------------------------------------------------
:30
--------------------------------------------------------------------------------
Please input the average daily truck traffic (ADTT) of the bridge.
* ADTT is the average daily truck traffic volume for the inventory route. ADTT
* for a bridge shall be compatible with other recorded items. For example, if
* the bridge roadway widths are recorded separately for parallel bridges, then,
* ADTT also needs to be the separated recorded values. Please refer to Items 29
* and 190 of Michigan Structure Inventory and Appraisal Coding Guide for
* details. In this software, ADTT value needs to be >= 100 & <= 30,000.
--------------------------------------------------------------------------------
:10000
--------------------------------------------------------------------------------
Please input the State County Code for the county where the bridge locates in.
* State County Code needs to conform with "Michigan Structure Inventory and
* Appraisal Coding Guide (page 9).
--------------------------------------------------------------------------------
:82
--------------------------------------------------------------------------------
Please input the approach surface type of the bridge.
          [1] Bitumen;
          [2] Mixed;
          [3] Concrete;
          [4] Unknown.
```

Figure F-7 Interface of SbNET 1.2 (predict using bridge design parameters)

```
--------------------------------------------------------------------------------
:2
--------------------------------------------------------------------------------
Please input the structural type of the bridge.
          [1] Simple/cantilevered steel bridge;
          [2] Continuous steel bridge;
          [3] Prestressed concrete bridge with I-girder;
          [4] Prestressed concrete bridge with adjacent box girder;
          [5] Prestressed concrete bridge with spread box girder;
--------------------------------------------------------------------------------
:1
--------------------------------------------------------------------------------

Ensemble of networks is simulating the bridge abutment condition ...

--------------------------------------------------------------------------------

--------------------------------------------------------------------------------

The abutment rating of the highway bridge in condideration is:
Abutment_Rating =

     8



--------------------------------------------------------------------------------

--------------------------------------------------------------------------------
Please input the type of the deterioration curve you would like to plot.
          [1] Deterioration curve based on exact predictions;
          [2] Deterioration curve based on smoothed predictions;
          [3] Deterioration curves based on both exact and smoothed predict

--------------------------------------------------------------------------------
:1
-------------------------------------------------------------------------------
The deterioration curve of the bridge in concern has been saved
in the folder as emf image
-------------------------------------------------------------------------------
----------------------------------------------------------------------------
Would you like to plot another another type of deterioration curve?

         [1] Yes;
         [2] No.
----------------------------------------------------------------------------
:2
----------------------------------------------------------------------------
Would you like to make prediction for another birdge?

         [1] Yes;
         [2] No.
----------------------------------------------------------------------------
```

Figure F-8 Interface of SbNET 1.2_continued (predict using bridge design parameters)

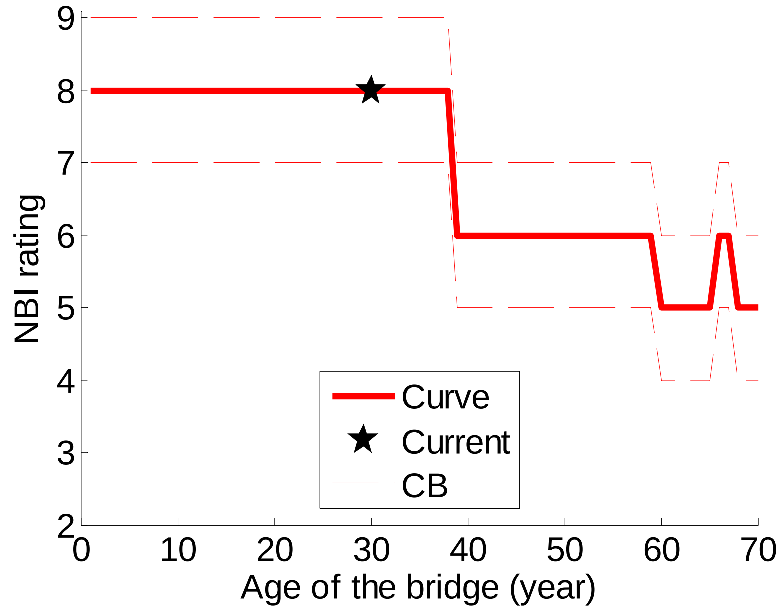


Figure F-9 An example of exact deterioration curve

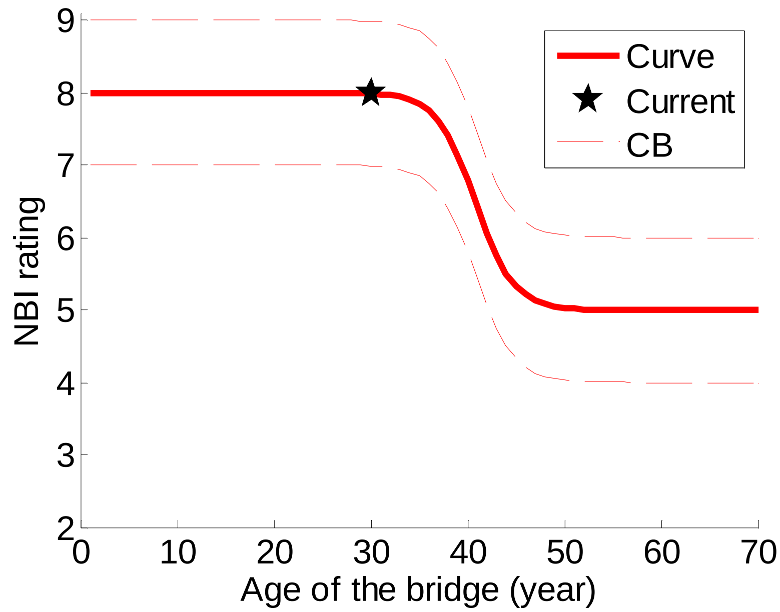


Figure F-10 An example of smoothed deterioration curve

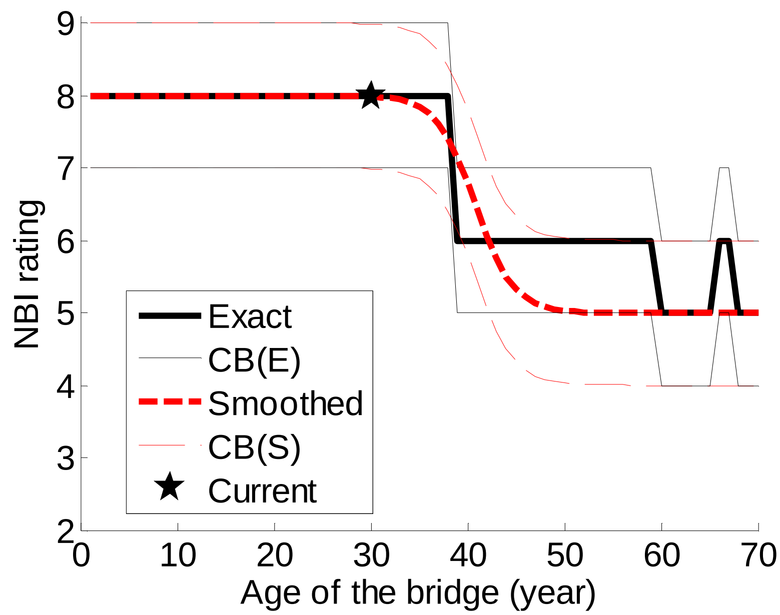


Figure F-11 An example of exact and smoothed deterioration curves

F.IV References

Burgueño, R. and Li, Z., "Identification of Causes and Development of Strategies for Relieving Structural Distress in Bridge Abutments," CEE-RR – 2008/02, Department of Civil and Environmental Engineering, Department of Civil and Environmental Engineering, Michigan State University, East Lansing, Michigan, February 2008.

Mathworks Inc., The, "Matlab R2007a," http://www.mathworks.com/, Natick, MA.